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DEVELOPMENT AND EVALUATION OF A MATHEMATICAL MODEL OF THE LOWER REACHES OF THE KANSAS RIVER DRAINAGE SYSTEM

Robert L. Smith, et al

Kansas University
Lawrence, Kansas

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Prepared Under Contract
PH 86-66-63
with the
Federal Water Pollution Control Administration
Missouri-Souris-Red River Basins Comprehensive Water Pollution Control Project

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Development and Evaluation of a Mathematical Model of the Lower Reaches of the Kansas River Drainage System

By

Robert L. Smith
Walter J. O'Brien
Albert R. LeFeuvre
Ernest C. Pogge

Civil Engineering Department
The University of Kansas

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Center for Research, Inc.
Engineering Science Division
Lawrence, Kansas

January, 1967

I

ABSTRACT

A mathematical model suitable for simulating the distribution of the conservative and the non-conservative water quality of the river system in the lower Kansas River basin is described. The model consists of three digital computer programs and considers both quasi-steady-state and dynamic hydrologic boundary conditions.

The hydrologic and conservative portion of the model has been tested for verification with discharge records for six time periods of from four to six weeks in length. Satisfactory verification was obtained. Additional refinement of the model could be achieved by increasing the scope of the data input but this does not appear to be justified at the present time.

Additional research will be necessary to more fully evaluate the parameters entering the non-conservative water quality model.

ACKNOWLEDGEMENT

The personnel of many agencies have contributed directly and indirectly to this study. However, special thanks are due Mr. Lewis A. Young, Project Director of the Missouri-Souris-Red River Basins Comprehensive Water Pollution Control Project, Federal Water Pollution Control Administration, and Mr. Melville W. Gray, Assistant Director of the Environmental Health Services Division, Kansas State Department of Health, for their enthusiastic cooperation and critical counsel.

The surface water discharge data used in developing the hydrologic relationships in the model were obtained from the appropriate Water Suppy Papers of the U. S. Geological Survey.

The discharge rating curves used to develop the velocity and hydraulic depth relationships used in the model were obtained from the Topeka District Office of the U. S. Geological Survey. The flood plain information study prepared by the Kansas City District Office of the U. S. Corps of Engineers was also helpful in this regard.

The time-of-flow determinations cited in the report were performed by the Federal Water Pollution Control Administration and the Environmental Health Services Division, Kansas State Department of Health.

The Environmental Health Services Division, Kansas State Department of Health, also collected the non-conservative water quality data cited in this report.

The chemical quality of the water samples collected during the summer of 1966 was analyzed by the Analytical Chemistry Laboratory of the State Geological Survey of Kansas. The chemical quality data for the sampling stations in existence prior to 1966 were obtained from the appropriate Water Supply Papers of the U. S. Geological Survey and the Bulletins of the Environmental Health Services Division, Kansas State Department of Health.

The mathematical model was developed on the facilities of the University of Kansas Computation Center. Mr. Alan Lumb, Mr. James Gossett, Mr. Donald Hey and Mr. Paul MacRoberts assisted with the development of the computer program.

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I Introduction

On January 15, 1966, The University of Kansas Center for Research, Inc., entered into contract number PH 86-66-63 with the Division of Water Supply and Pollution Control, Federal Water Pollution Control Administration, for the development and evaluation of a mathematical model of the streamflow in the lower reaches of the Kansas River basin. The completed model was to be capable of:

(1) Simulating the hydrologic and hydraulic behavior of the river system;

(2) Predicting the time history and distribution of conservative and non-conservative pollutants throughout the river system; and

(3) Predicting the location of critical areas of pollution under various types of river basin development.

Three computer programs have been developed in fulfillment of the contract. One program considers the river system under quasi-steady-state conditions and simultaneously evaluates both the conservative and the non-conservative water quality throughout the model area. The other two programs are dynamic in that they utilize input hydrographs for boundary conditions and predict the average daily water quality throughout the system. One of the dynamic programs treats the problem of conservative pollutants; the other is concerned with non-conservative pollutants. The hydrology and hydraulics of all of the programs are identical. The decision to develop separate dynamic programs for the two classes of pollutants was prompted by limitations in computer storage capacity.

All of the programs will consider withdrawals of water for use within the area and will account for consumptive use within the river basin. The programs have widespread applicability for both long range planning purposes and short range reservoir

reservoir operational needs. Pertinent examples are the determination of the probable effect on water quality in the lower basin of projected future expansion in irrigation development in the upper basin, and the determination of the sequence of reservoir releases required to optimize the water quality in the lower basin during drought periods.

The purpose of this report is to describe each of the programs in detail. This is accomplished by first introducing the model region and then outlining the hydrologic and hydraulic concepts; the factors considered in the prediction of conservative water quality; and the techniques used in modeling the non-conservative water quality. Flow-through diagrams for the computer programs are described in detail and the results obtained in the verification studies are presented and discussed. Appropriate information on the computer programs will be found in the appendices.

During the course of this investigation it became apparent that the biological, the chemical, and the time-of-flow data available for the model region were insufficient to establish many of the parameters required to accurately describe the drainage system. A series of investigations was therefore undertaken to correct this situation. The data obtained from these studies is referred to where appropriate. However, the primary emphasis in this report is on the mathematical model.

II. The Model Region

The Kansas River basin contains about 60,060 square miles and lies within the states of Colorado, Nebraska, and Kansas. Approximately eighty-five per cent of the area is semiarid. The eastern fifteen per cent of the basin is classified as humid.

The rivers within the basin flow in a predominantly eastward direction and culminate in the Kansas River, which is formed by the junction of the Republican River and the Smoky Hill River near Junction City, Kansas. The Kansas River empties into the Missouri River at Kansas City.

The portion of the basin included in the present model is indicated by the shaded area in Figure 1, and contains approximately 8,820 square miles. It is bounded by six reservoirs, as indicated in Figure 2. The reservoir storage capacities and the average annual flow in the rivers at the reservoir sites are given in Table 1. The last column in this table lists the reductions in yield which are anticipated from future irrigation developments in the basin (1).

A significant segment of the industrial economy of Kansas is located within the area. This activity is concentrated primarily in the lower Kansas River valley below Topeka. Large amounts of wheat and corn and large numbers of beef cattle also are produced throughout the model area. The populations of the major cities are given in Table 2. Comprehensive reviews of the water resources problems of the region are available in a series of reports published by the Kansas Water Resources Board (2) (3) (4).

Table 1
Characteristics of the Major Rivers and Reservoirs
of the Lower Kansas River Basin (1)

River	Reservoir	Date of Reservoir Completion	Conservation plus Sediment Storage and Total Storage acre feet	Average Annual Flow Before Regulation CFS	Anticipated Flow Decrease Per Cent
Delaware	Perry	June, 1969	290,000 770,000	430	——
Big Blue	Tuttle Creek	March, 1962	425,300 2,367,000	1,700	15
Republican	Milford	June, 1966	460,000	1,170	29
Solomon	Glen Elder	June, 1970	239,100 976,000	350	25
Saline	Wilson	June, 1965	265,000 775,000	135	3 [2]
Smoky Hill	Kanopolis	February, 1948	47,300 432,900 [1]	290	10

[1] Current (1966) storage adjusted to include sediment accumulation since construction.

[2] Irrigation development below Wilson Reservoir will cause an anticipated 26% flow decrease at Tescott on the Saline River.

Table 2

1965 Population of the Major Cities Located in the Lower Kansas River Basin (5)

Junction City	20,975	Overland Park	56,395
Kansas City	123,449	Prairie Village	29,738
Lawrence	27,028	Roeland Park	10,430
Manhattan	22,716	Shawnee	14,989
Mission	8,325	Salina	38,706
Olathe	12,661	Topeka	125,363

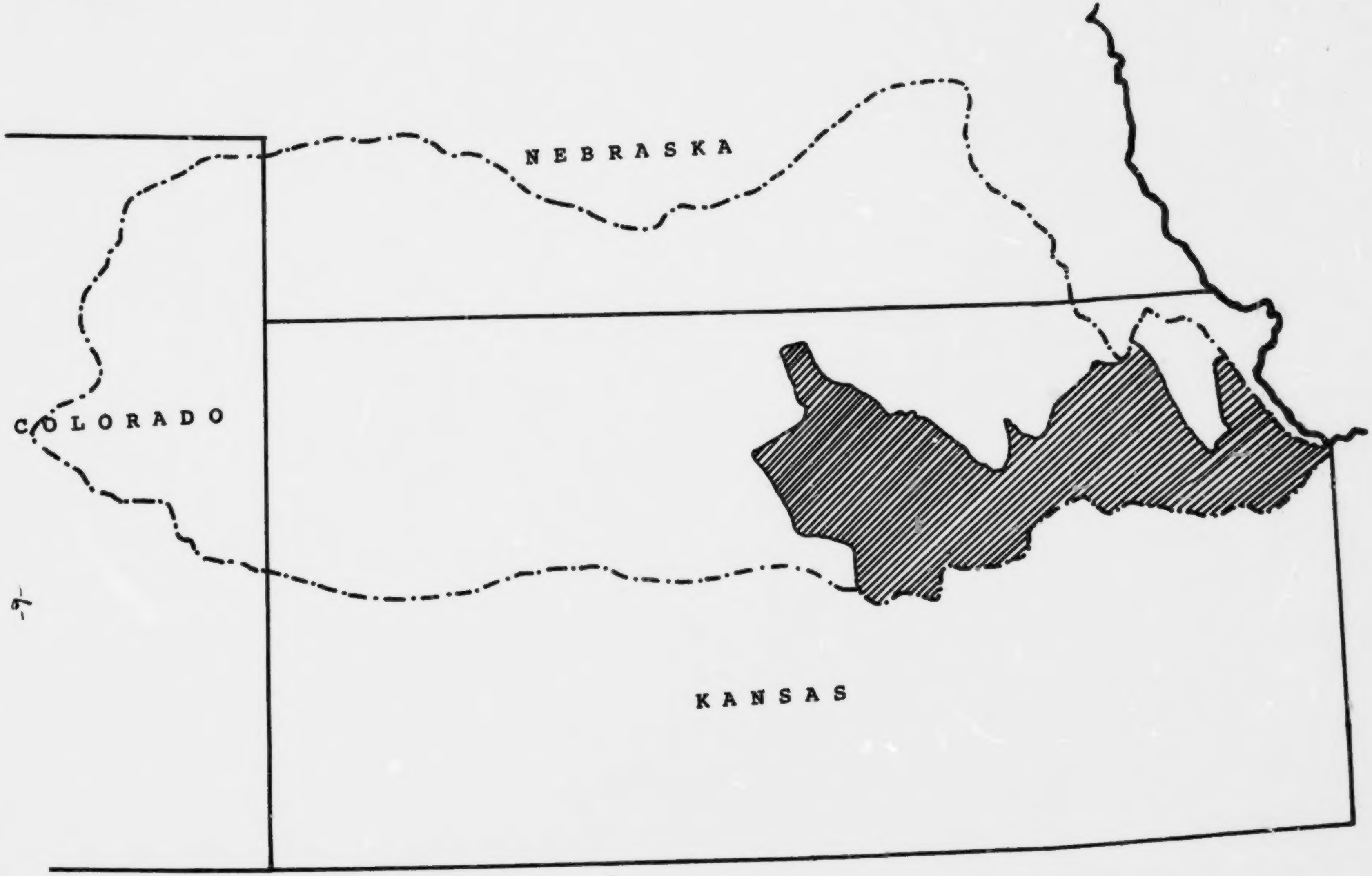
NEBRASKA
COLORADO
KANSAS

Figure 1 Location Map of the Lower Kansas River Basin Model Area

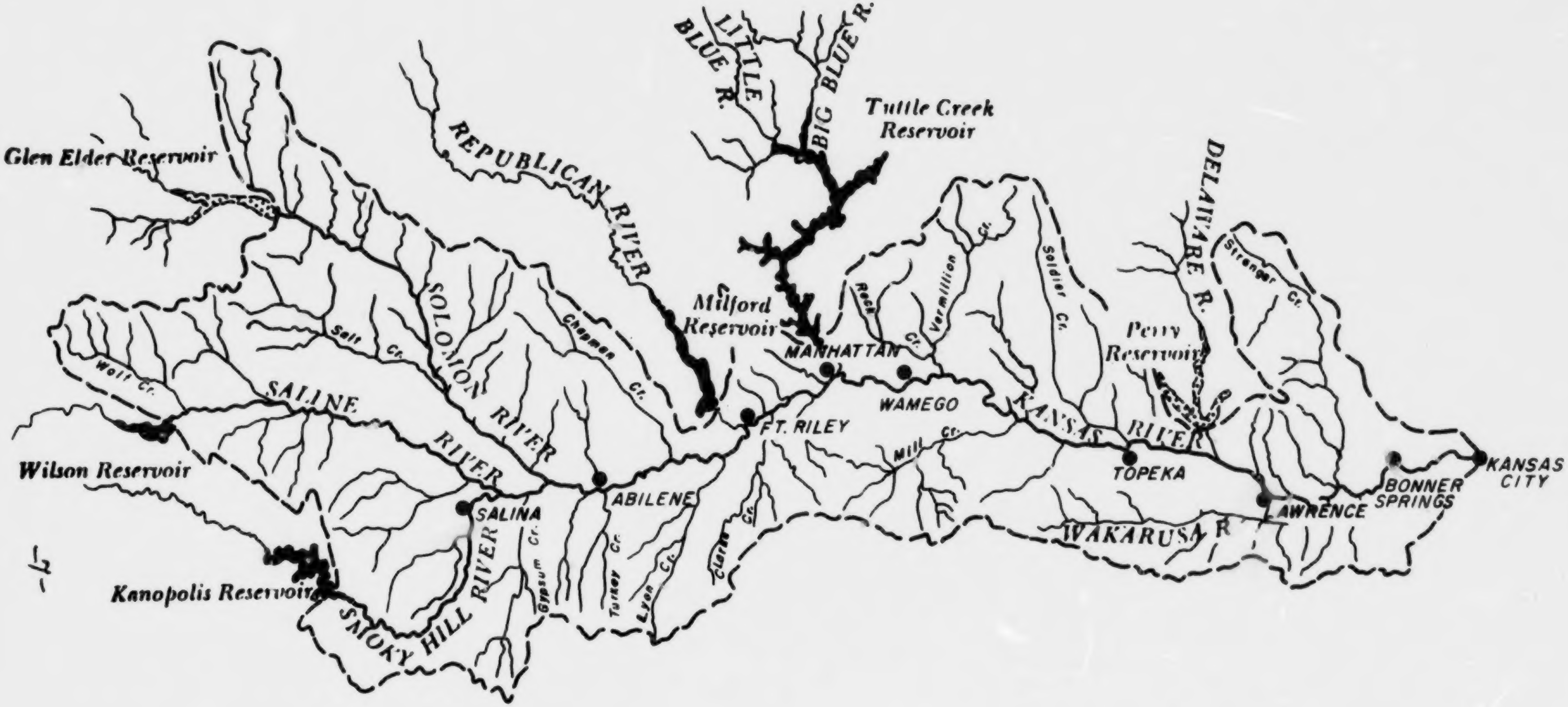
Glen Elder Reservoir
REPUBLICAN RIVER
LITTLE BLUE R.
BIG BLUE R.
Tuttle Creek Reservoir
DELAWARE R.
Strenger Cr.
Perry Reservoir
Soldier Cr.
Vermillion Cr.
Rock Cr.
Milford Reservoir
MANHATTAN
WAMEGO
KANSAS RIVER
FT. RILEY
Mill Cr.
TOPEKA
LAWRENCE
BONNER SPRINGS
KANSAS CITY
WAKARUSA R.
Chapman Cr.
SOLOMON RIVER
Salt Cr.
Wolf Cr.
SALINE RIVER
Wilson Reservoir
SALINA
ABILENE
Gypsum Cr.
Turkey Cr.
Lyon Cr.
Clarks Cr.
SMOKY HILL RIVER
Kanopolis Reservoir

Figure 2   Physical Details of the Lower Kansas River Basin Model Area

III. Hydrology and Hydraulics of the Region

The topography of the portion of the Kansas River basin included in the model study area is primarily of two types. In the eastern section it is depositional in nature as a result of glaciation. The post-glacial topography, however, has been modified considerably by erosion. Adjacent to the larger streams the land is highly dissected and the topography is rough to hilly. The topography of the western section is more rock-controlled, with irregular, east-facing escarpments and considerable dissection. There still exist broad reaches of flat uplands. Some of the streams in this section have steep-sided narrow canyons, and the relief may be in excess of 300 feet. The major streams of the model area have developed fairly wide valleys which are underlaid by alluvium and terrace deposits.

The average annual precipitation in the study area varies from 24 inches in the western portion to 36 inches near the mouth of the Kansas River. Practically all of the winter precipitation and much of the summer precipitation in this area takes place along the line of contact between cold air masses moving down from the northwest and warm, moist air moving up from the Gulf of Mexico. This results in a southwest - northeast orientation of the storm axis. Rainstorms in this area may be of the thunderstorm or the continental storm type. Average annual snowfall varies from 18 to 21 inches. On the average, the ground is snow-covered less than one month per year. July is normally the warmest month, with an average daily maximum of 92°F and minimum of 67°F. January is the coldest month, with an average daily maximum and minimum of 40°F and 18°F respectively. The growing season averages 175 days. The average annual evaporation from reservoir surfaces is about 45 inches, with a range of from 40 to 60 inches.

The outflow from the portion of the Kansas River basin included in the model study is derived primarily from four sources. These are: controlled releases from the reservoirs which form the boundary of the study area, discharge from the numerous tributary streams in the area, inflow from local areas tributary to the main streams, and ground-water effluent from alluvial and terrace deposits adjacent to the main streams. This latter source is especially significant in the lower portion of the Kansas River. During low-flow periods, the major portion of the discharge in the Kansas River is sustained by releases from the reservoirs. During the periods of appreciable runoff, however, the tributary streams can become the major contributors to the flow in the Kansas River. The ground-water effluent becomes relatively significant during low-flow periods following hydrologic events which have resulted in considerable recharge to the alluvial aquifers. For example, high-stage flows create a large amount of bank storage, or large amounts of infiltration accumulate in the alluvium from extended rains.

The Water Resources Division of the U. S. Geological Survey maintains a network of stream-gaging stations throughout the Kansas River basin. These stations are located on tributary streams as well as on the main streams in the basin. The thirty-two stations shown in Figure 3 were used in this model study. The gaging stations on the tributary streams are not located at the mouths of the streams, but are located some distance upstream from the junction with the larger stream to avoid backwater effects in the stage-discharge relation. As a result, the total drainage area of a tributary stream is not gaged. The effect this has on the prediction of the amount of flow being contributed by each tributary is apparent from Figure 4. The shaded areas are the portions of the tributary drainage areas which are gaged, and in every case a portion of the total drainage area is not included. A significant number of the tributaries in the study area are not gaged. Table 3 lists the stream gaging station in the study area according to the numerical designation used in Figure 3, together with station name, stream name, period of record, gaged area, and total drainage area of the stream at the mouth. Stream-gaging stations have been installed immediately downstream from all of

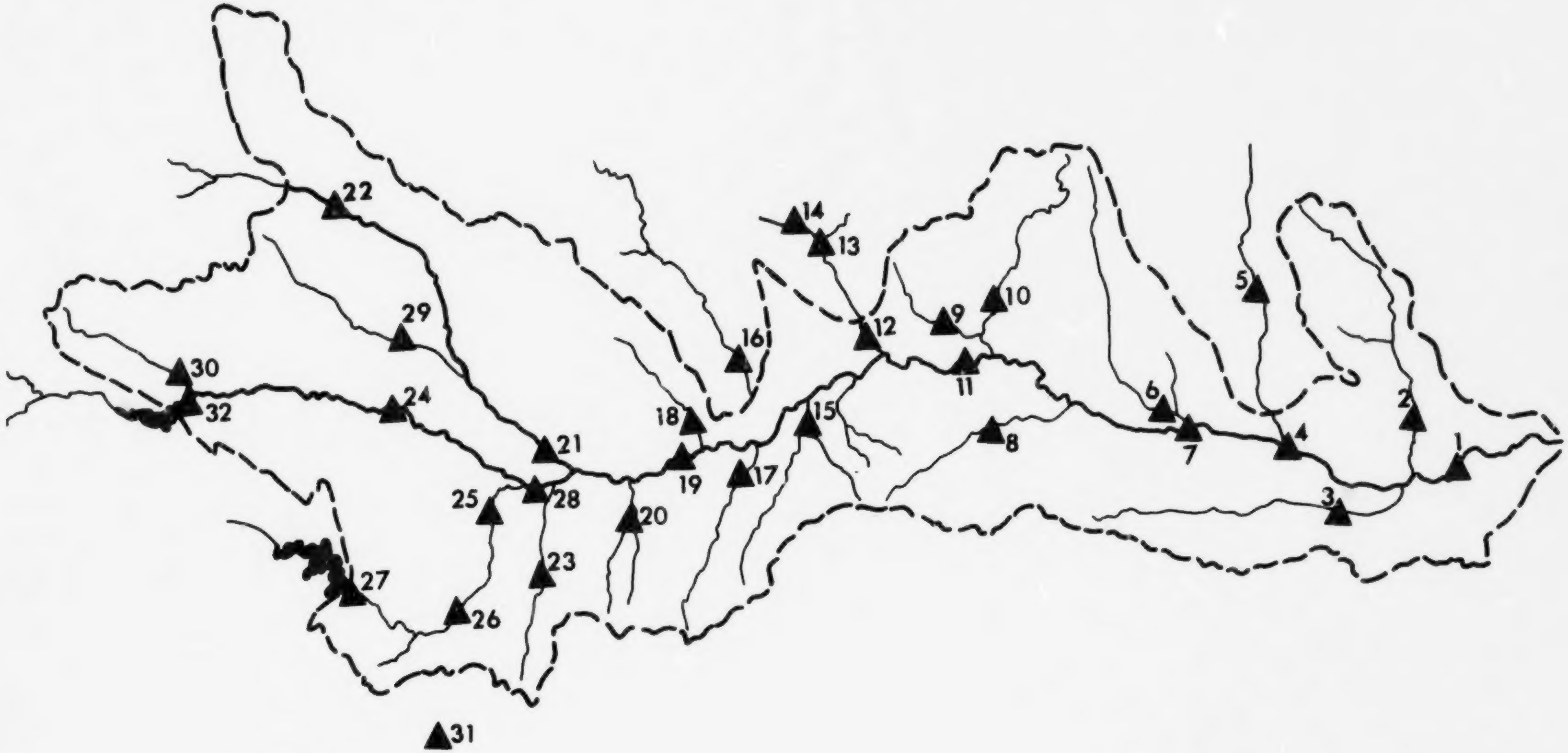
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Figure 3 Discharge Gaging Stations Used in Developing the Lower Kansas River Basin Model

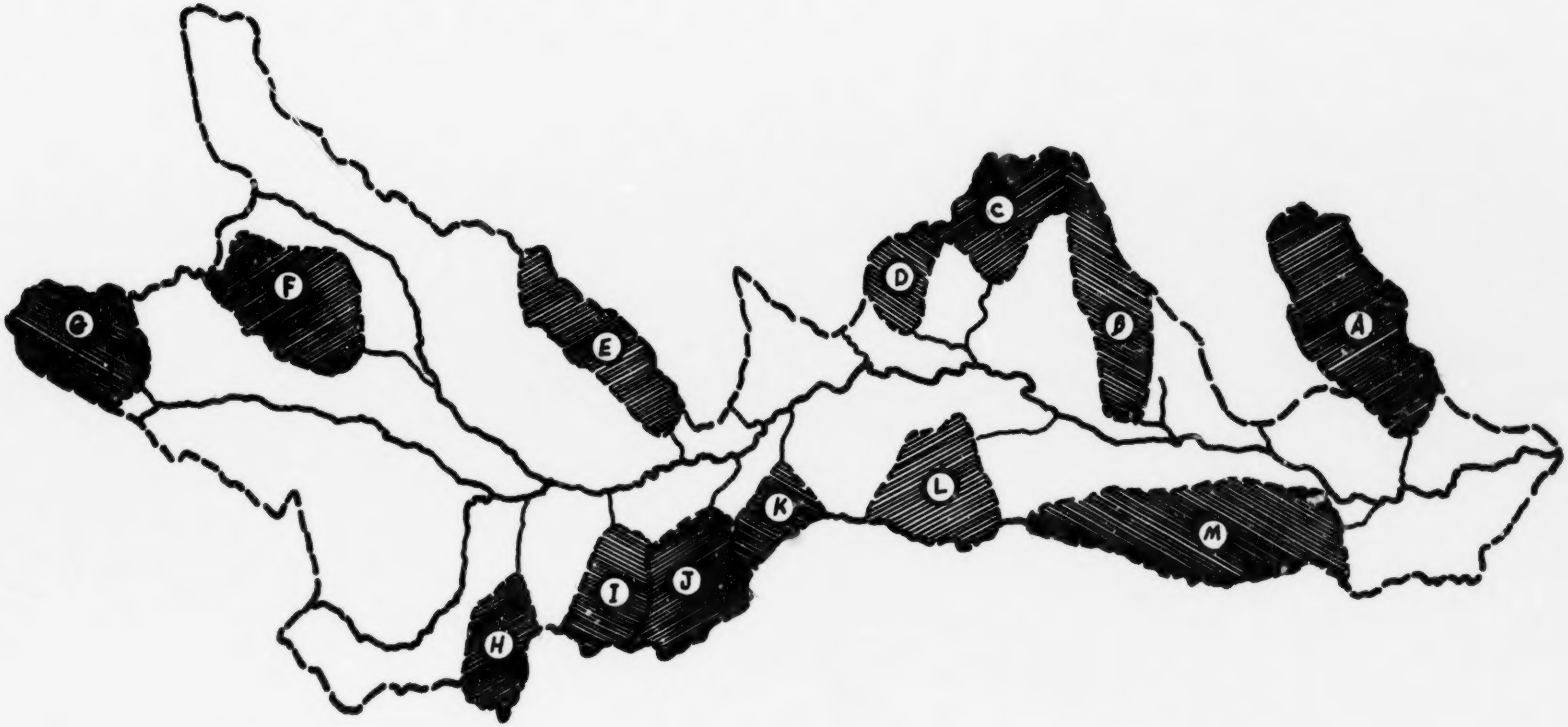
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Figure 4 Primary "Q-Add" Areas used in Developing the Lower Kansas River Basin Model

Table 3
Stream Gaging Station Used In The Lower Kansas River Model

No.	Station Name	Stream	Period of Record	Gaged Area sq. mi.	Total Area at Mouth sq. mi.
1	Bonner Springs	Kansas River	1917 - 1966	59,890	——
2	Tonganoxie	Stranger Creek	1929 - 1966	406	530
3	Lawrence	Wakarusa River	1929 - 1966	425	521
4	Lecompton	Kansas River	1936 - 1966	58,420	——
5	Valley Falls	Delaware River	1922 - 1966	922	1,159
6	Topeka	Soldier Creek	1935 - 1966	290	332
7	Topeka	Kansas River	1917 - 1966	56,710	——
8	Paxico	Mill Creek	1953 - 1966	316	408
9	Louisville	Rock Creek	1958 - 1965	128	184
10	Wamego	Vermillion Creek	1936 - 1946 1954 - 1966	243	517
11	Wamego	Kansas River	1919 - 1966	55,240	——
12	Manhattan	Big Blue River	1954 - 1966	9,560	9,615
13	Randolph	Big Blue River	1918 - 1960	9,100	9,615
14	Winkler	Fancy Creek	1953 - 1966	176	289
15	Junction City	Clark Creek	1957 - 1965	200	280
16	Milford	Republican River	1950 - 1960	24,900	——
17	Woodbine	Lyon Creek	1953 - 1966	230	283
18	Chapman	Chapman Creek	1953 - 1966	300	330
19	Enterprise	Smoky Hill River	1934 - 1966	19,200	——
20	Abilene	Turkey Creek	1958 - 1965	143	167

Table 3
(Continued)
Stream Gaging Stations Used In The Lower Kansas River Model

No.	Station Name	Stream	Period of Record	Gaged Area sq. mi.	Total Area at Mouth sq. mi.
21	Niles	Solomon River	1917 - 1966	6,770	6,856
22	Beloit	Solomon River	1929 - 1965	5,530	6,856
23	Gypsum	Gypsum Creek	1954 - 1966	120	263
24	Tescott	Saline River	1919 - 1966	2,820	3,464
25	Mentor	Smoky Hill River	1923 - 1930 1947 - 1966	8,230	——
26	Lindsborg	Smoky Hill River	1930 - 1965	8,110	——
27	Langley	Smoky Hill River	1940 - 1966	7,857	——
28	New Cambria	Smoky Hill River	1962 - 1966	11,730	——
29	Ada	Salt Creek	1959 - 1966	384	466
30	Lucas	Wolf Creek	1959 - 1966	163	294
31	Valley Center	Little Arkansas River	1922 - 1966	1,327	——
32	Wilson	Saline River	1929 - 1966	1,900	——

the completed reservoirs, and the discharges observed at these stations can be assumed to be the discharges released from the reservoirs. At the present time these include Kanopolis, Wilson, and Tuttle Creek Reservoirs.

Station 31, Little Arkansas River at Valley Center, Kansas, is not located in the Kansas River basin. The reason for its being included in this study will be discussed later.

A complete discussion of the mathematical model is given in Chapter VI, but to understand the hydrologic and hydraulic needs of the model, it is necessary to briefly examine these portions of the model at this point. The stream system in the model area is broken down into a series of reaches, as indicated in Figure 18. The end points of these reaches are determined by gaging station locations, point sources of pollution, reservoirs, junctions of gaged tributary streams with the main streams and confluences of the main streams. Beginning with the input values from the reservoirs at the western boundary of the region, the model applies a mathematical accounting procedure for both quantity and quality as it moves through the stream system, until the last reach at the mouth of the Kansas River is included. For each reach, the inflow from the upstream reach is determined, and the added inflow from other sources, such as ungaged tributaries and local drainage, is predicted.

The model requires, as initial boundary value inputs, the discharge values for the uppermost reaches in the stream system. Where the stream begins at a reservoir, the discharge for the initial reach is the discharge released from the reservoir, and is determined from the stream-gaging station immediately below the reservoir. For gaged tributaries not having reservoirs, the discharge in the initial reach is the discharge observed at the gaging station. For all reaches other than the initial reaches, the model requires a prediction of the discharge added to the reach from local sources. This increase in discharge is designated QADD. For the determination of time-of-travel through the reach as well as for the examination of quality changes within the reach, the average velocity and average hydraulic depth are required as a function of the discharge in the reach.

The prediction of QADD values involves the determination of discharges to the various reaches of the model from ungaged portions of the basin. The QADD discharges were related to the input parameters of the model, which were the observed daily discharges at the stream-gaging stations on the gaged tributaries. The discharge from the ungaged portion of the drainage area of the gaged tributaries is predicted by multiplying the observed discharge at the gaging station by the ratio of ungaged area to gaged area.

To determine the discharge from ungaged tributaries, cross-correlations were made for observed discharges at gaging stations in the area of the ungaged tributary. Monthly mean discharges were used in these correlations, and the direction of best correlation relative to the ungaged tributary was determined. The direction of best correlation was influenced by the fact that most storms in the area follow a southwest-to-northeast axis. The discharge of the ungaged tributary is predicted by multiplying the observed discharge at the gaging station which gave the best directional correlation by the ratio of ungaged tributary area to gaged area. In some cases the sum of the observed discharges for two gaging stations is used.

The local discharge from the ungaged drainage areas adjacent to the major streams was obtained by determining the gain in each of the reaches between gaging stations on the major stream. The contribution of the tributaries to each of the reaches was subtracted from the gain in the reach. The residual gain was correlated with tributary gaging-station discharges in the vicinity of the particular reach. The predicted local inflow is obtained by multiplying the observed discharges at a tributary gaging station or the sum of the observed discharges at two tributary gaging stations by a factor obtained from both the correlation analysis and a comparison of gaged drainage area with ungaged drainage area.

In the upstream reaches of the area, the ground water effluent does not comprise a significant amount of the discharge in the reach.

For the reaches of the Kansas River below Lecompton, this is not true during periods of relatively low discharge. To compensate for the groundwater contribution to these reaches, a constant inflow per mile of stream length was introduced into the QADD functions. This constant inflow value was determined by calculating the gain in each of the reaches and subtracting from this value the predicted inflows from surface water sources. The residual was assumed to be derived from groundwater sources. An attempt was made to correlate these residual values with observed discharges at adjacent tributary gaging stations, but no significant relationship was obtained.

The manner in which the QADD functions operate in the model will be discussed in Chapter VI. The QADD functions for each of the reaches are tabulated in Appendix E.

Not all of the thirty-one gaging stations listed in Table 3 were used either as boundary input indicators or in the QADD functions. The gaging stations on the Kansas River and those on the other major streams in the area below the stations at the reservoirs were used only in the verification of the model.

The verification of the model was performed by using historic data from all of the gaging stations in the area. This necessitated some changes in the boundary value input procedure, because not all six of the reservoirs were in operation during the period of the historic record used.

Perry Reservoir has not been completed. The discharge from this reservoir has been simulated by using the discharge at Gaging Station 5 and developing a QADD function for Reach 40 on the basis of the discharge at Gaging Station 2. When Perry Reservoir is in operation, this routine will be replaced by the actual releases from the reservoir. Tuttle Creek Reservoir was not in operation during the period of record used in the model verification, and its releases were simulated by the discharges at Gaging Stations 13 and 14 and a QADD function based on the discharge at Gaging Station 14. Tuttle Creek Reservoir is now in operation, and the discharge at Gaging Station 12 can be used as the release from the reservoir.

Milford Reservoir was not in operation, and its releases were simulated by the discharge at Gaging Station 16 and a QADD function for Reach 130 based on the discharge at Gaging Station 18. Glen Elder Reservoir has not been completed. Its releases were simulated by adjusting the discharge values at Gaging Station 22 by the difference in drainage area. The discharges at the stations at Wilson Reservoir and Kanopolis Reservoir were used for the releases from these two reservoirs.

In one instance it was necessary to subdivide a tributary stream above the gaging station. Mill Creek above the gaging station at Paxico, Station 8, was divided into two reaches to bring in a point source of pollution from the city of Alma. The discharge at Paxico was distributed to the reaches on the basis of contributing drainage area.

In the western portion of the area, the areal density of tributary gaging stations is much smaller than in the eastern portion. As a result, it was necessary to use a gaging station outside of the Kansas River Basin, Station 31, to predict the QADD for three of the reaches of the Smoky Hill River just below Kanopolis Reservoir. With relatively large drainage areas in the western portion assigned to each of the tributary gaging stations for QADD predictions, the probability of failing to predict appreciable amounts of runoff from local thunderstorms becomes significant.

The major problem in the eastern portion of the area is the method of predicting the groundwater contribution to the lower reaches of the Kansas River. A constant inflow per unit length is the simplest and also probably the least accurate method. A more accurate approach would require additional basic data, such as groundwater levels in the alluvial aquifers, and would further complicate the mathematical model.

The velocity and hydraulic depth data for each of the reaches is given in Appendix E. The velocity and hydraulic depth for each reach are computed from linearized functions obtained from log-log graphs of velocity versus discharge and hydraulic depth versus discharge. The data for these graphs was obtained from the discharge rating curves and discharge measurements for the stream-gaging stations operated by the U. S. Geological Survey. Figure 5 is a typical example of one of these graphs. The accuracy of the velocity relationship is being tested by time-of-flow measurements at the present time. The triangular points shown in Figure 5 are the results of some of these measurements.

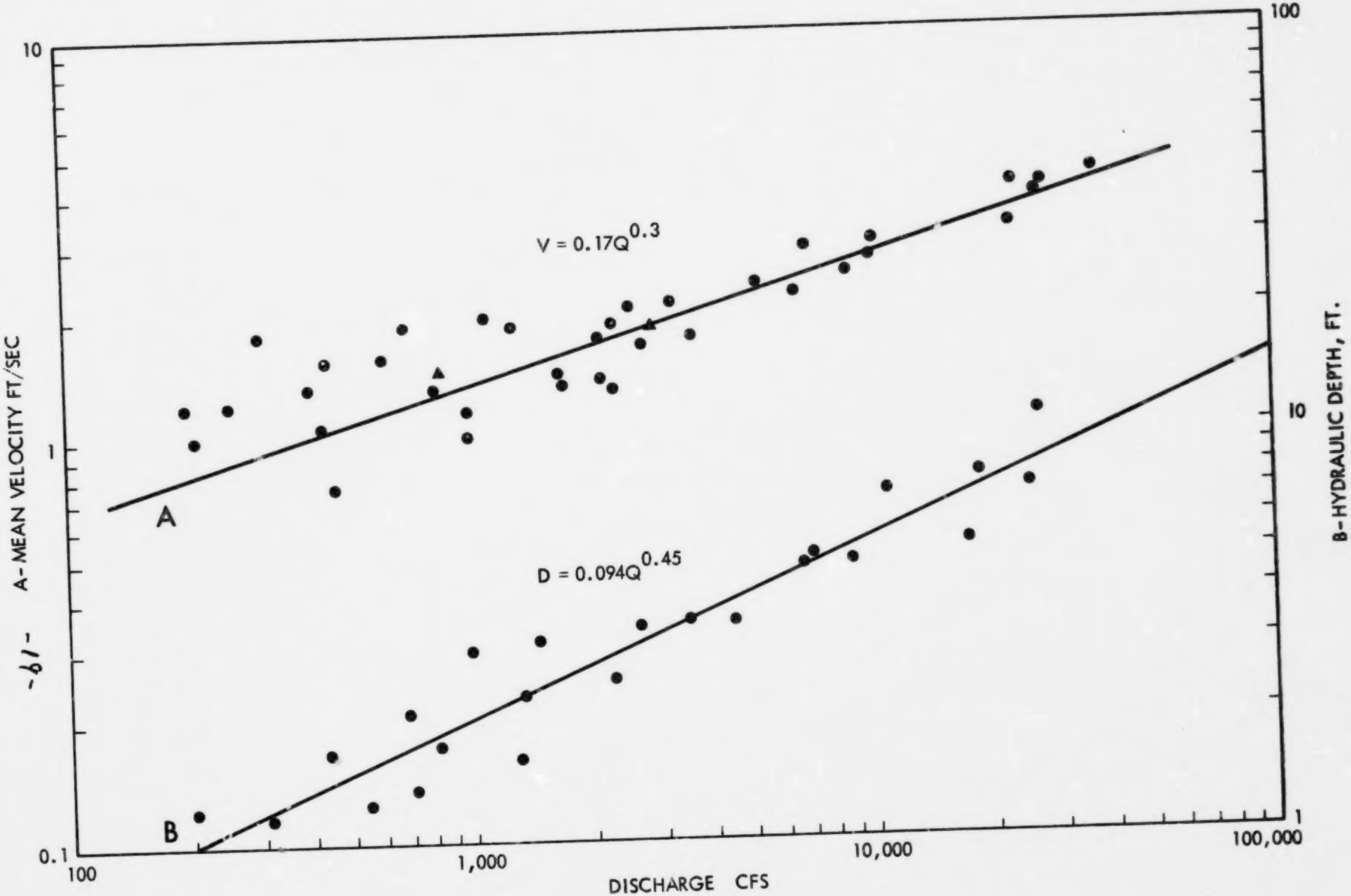


Figure 5 Mean Velocity and Hydraulic Depth as a Function of Discharge for the Kansas River at Topeka

IV. Basin Geology and Conservative Water Quality

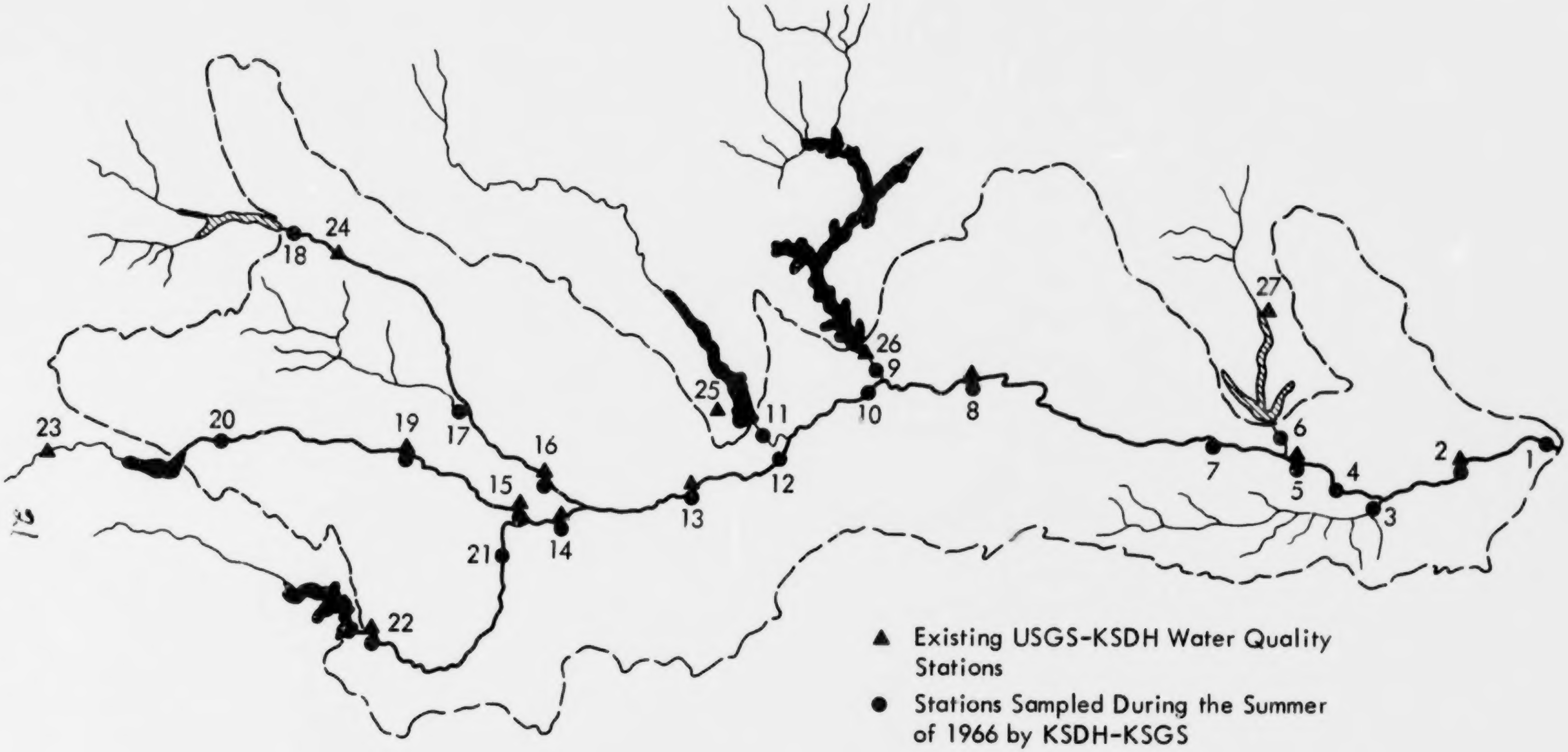
The geology of the Lower Kansas River basin is sedimentary in nature; and for purposes of water quality characterization, the area can be divided into three general regions.

Rocks of Cretaceous age predominate in the area west of the junction of the Solomon and Smoky Hill Rivers. A significant unit in this area is the Dakota Formation, which contains zones of a highly saline nature. Drainage from this area is thought to supply most of the dissolved solids, particularly chloride, originating in the lower reaches of the Solomon, the Saline, and the Smoky Hill Rivers.

The surface geology between the junction of the Solomon and the Smoky Hill Rivers and the town of Wamego on the Kansas River consists primarily of rocks of Permian age. These include a series of shales, limestones, and sandstones, all of which contain relatively large amounts of gypsum. This region contributes significant amounts of calcium, sulfate, carbonate, and some chloride to the river system.

The surface geology east of Wamego consists primarily of rocks of Pennsylvanian age. There are fewer bedrock outcrops north of the Kansas River, however, because of the loess and till mantle left by quaternary glaciation. Significant amounts of calcium and carbonate enter the river system in this region. Relatively little chloride originates in this area.

The network of regular water-quality sampling stations operated jointly by the United States Geological Survey and the Kansas State Health Department, and the supplemental water-quality stations operated by the Kansas State Health Department during the summer of 1966, are shown in Figure 6. A detailed description of the sites is given in Table 4. Chemical water quality data are available for all 27 stations in the network. Mathematical analysis of all of the spot sample data available from these stations has been conducted in an effort to characterize the water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▲ Existing USGS-KSDH Water Quality Stations
● Stations Sampled During the Summer of 1966 by KSDH-KSGS

Figure 6 Location Map of the Water Quality Sampling Stations Used in Developing the Lower Kansas River Basin Model

Table 4

Water Quality Stations Used in The Lower Kansas River Model

No.	Station Name	Stream	River Mile	Period of Record	USGS Station No.
1	Kansas City	Kansas River	0.3	Summer 1966	—
2	Bonner Springs	Kansas River	20.3	1961 - 1964 Summer 1966	6-8923
3	Eudora	Wakarusa River	41.7-0.4	Summer 1966	—
4	Lawrence	Kansas River	53.1	Summer 1966	—
5	Lecompton	Kansas River	64.1	1957 - 1958 1961 - 1964 Summer 1966	6-8910
6	Perry	Delaware River	64.7-1.8	Summer 1966	—
7	Topeka	Kansas River	84.0	Summer 1966	6-8890
8	Wamego	Kansas River	129.1	1956 - 1958 1961 - 1964 Summer 1966	6-8875
9	Casement Bridge	Big Blue River	147.1-7.3	Summer 1966	—
10	Manhattan	Kansas River	148.3	Summer 1966	—
11	Milford	Republican River	170.4-6.0	Summer 1966	—
12	Junction City	Smoky Hill River	170.4-2.0	Summer 1966	—

Table 4
(Continued)

No.	Station Name	Stream	River Mile	Period of Record	USGS Station No.
13	Enterprise	Smoky Hill River	170.4-43.3	1955 - 1958 1961 - 1964 Summer 1966	6-8776
14	New Cambria	Smoky Hill River	170.4-83.7	1962 - 1964 Summer 1966	6-8776
15	New Cambria	Saline River	170.4-89.6-2.0	1962 - 1964 Summer 1966	6-8701
16	Niles	Solomon River	170.4-68.3-16.9	1958 - 1964 Summer 1966	6-8769
17	Minneapolis	Salt Creek	170.4-68.3-45.9-10.4	Summer 1966	—
18	Glen Elder	Solomon River	170.4-68.3-155.1	Summer 1966	—
19	Tescott	Saline River	170.4-89.6-68.5	1949 - 1953 1961 - 1964 Summer 1966	6-8695
20	Sylvan Grove	Saline River	170.4-89.6-113.5	Summer 1966	—
21	Mentor	Smoky Hill River	170.4-117.0	1963 - 1964 Summer 1966	6-8665
22	Langley	Smoky Hill River	170.4-177.0	1961 - 1964 Summer 1966	6-8655

Table 4
(Continued)

No.	Station Name	Stream	River Mile	Period of Record	USGS Station No.
23	Russell	Saline River	170.4-89.6-167.9	1946 - 1949 1961 - 1964	6-8670
24	Beloit	Solomon River	170.4-68.3-130.6	1949 - 1952 1957 - 1958 1959 - 1964	6-8760
25	Milford	Republican River	170.4-18.6	1955 - 1958 1961 - 1964	6-8570
26	Manhattan	Big Blue River	147.1-9.7	1955 - 1958 1961 - 1964	6-8870
27	Valley Falls	Delaware River	64.7-30.6	1961 - 1964	6-8905

type and variability, and to develop prediction equations suitable for monitoring, on a continuous basis, changes in the water quality (6). Graphical analyses of the correlation between dissolved ionic concentration and discharge also were conducted for all of the stations for which discharge measurements were available. However, the drought which occurred during the summer of 1966 restricted the final inorganic water quality network used in the model to 12 of the longer-term United States Geological Survey Stations in the basin. The 1966 summer data are being used to refine the records on these stations in the low discharge regions and to pinpoint the areas where a change in water quality occurs.

A significant portion of the inorganic load carried by the stream network originates within the model area. This is illustrated in Figure 7, which shows the average chloride load in tons per day for the 12 longer-term stations and the average increase in chloride load between adjacent stations. These values were computed by integrating the discharge-versus-chloride-concentration curve with the flow-duration curve for each water-quality station. An example of one of these integrations is given in Table 5. The delta loads shown in the regions between stations represent the average weight of dissolved chloride ion which enters the river system with the QADD discharge.

Equations describing the increase in conservative pollutant load from natural sources can be developed for each individual ion species of interest and the QADD discharge in the model. A procedure for developing these equations is given in Figures 8 through 11. In this illustration the discharge-versus-chloride-concentration curves for the Solomon River at Beloit and Niles were developed from the observed data. These graphs were converted to discharge-versus-chloride-load curves, and then related to each other by the discharge correlation obtained between the two stations in Figure 10. The changes in chloride load associated with selected incremental changes in QADD were then graphed to form the plot illustrated in Figure 11. The equation of this

Table 5

Computation of the Average Chloride Load for the Solomon River at Niles - Quality Station Number 16

Flow Duration %	Discharge CFS	Chloride Concentration mg/L	Chloride Load Tons/Day	Interval Average
0.1	20,310	1.3	70.1	
1.0	7,447	4.7	93.5	81.8
2.0	4,942	7.9	105.9	99.7
5.0	2,302	22.0	136.8	121.4
10.0	1,016	58.0	158.9	147.9
			Weighted Average	127.7
10.0	1,016	58	158.9	
20.0	433	130	152.1	155.5
30.0	284	168	128.9	140.5
40.0	203	195	106.9	117.9
50.0	149	216	86.8	96.9
60.0	115	228	70.9	78.9
70.0	88	238	56.6	63.8
80.0	62	250	41.8	49.2
90.0	39	270	28.3	35.1
90.0	39	270	28.3	
95.0	28	282	21.4	24.9
98.0	20	300	16.2	18.8
99.0	16	308	13.2	14.7
99.9	9	320	7.7	10.5
			Weighted Average	20.5

Accumulative Total 885.6

Average Discharge $\frac{885.6}{10}$ = 88.6 Tons/Day

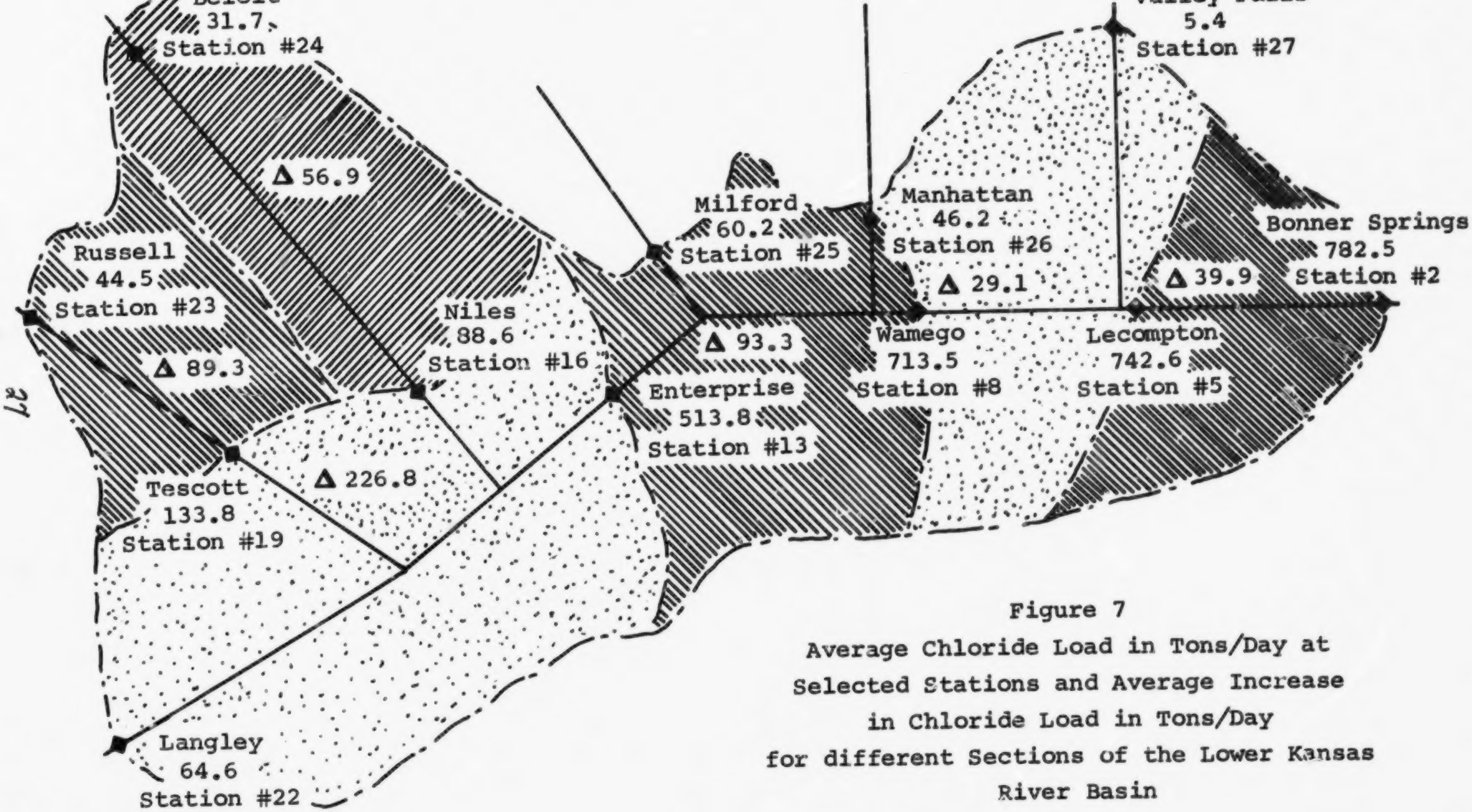

Beloit
31.7
Station #24
Δ 56.9
Russell
44.5
Station #23
Δ 89.3
Niles
88.6
Station #16
Tescott
133.8
Station #19
Δ 226.8
Langley
64.6
Station #22
Milford
60.2
Station #25
Δ 93.3
Enterprise
513.8
Station #13
Manhattan
46.2
Station #26
Δ 29.1
Wamego
713.5
Station #8
Valley Falls
5.4
Station #27
Δ 39.9
Lecompton
742.6
Station #5
Bonner Springs
782.5
Station #2

Figure 7

Average Chloride Load in Tons/Day at
Selected Stations and Average Increase
in Chloride Load in Tons/Day
for different Sections of the Lower Kansas
River Basin

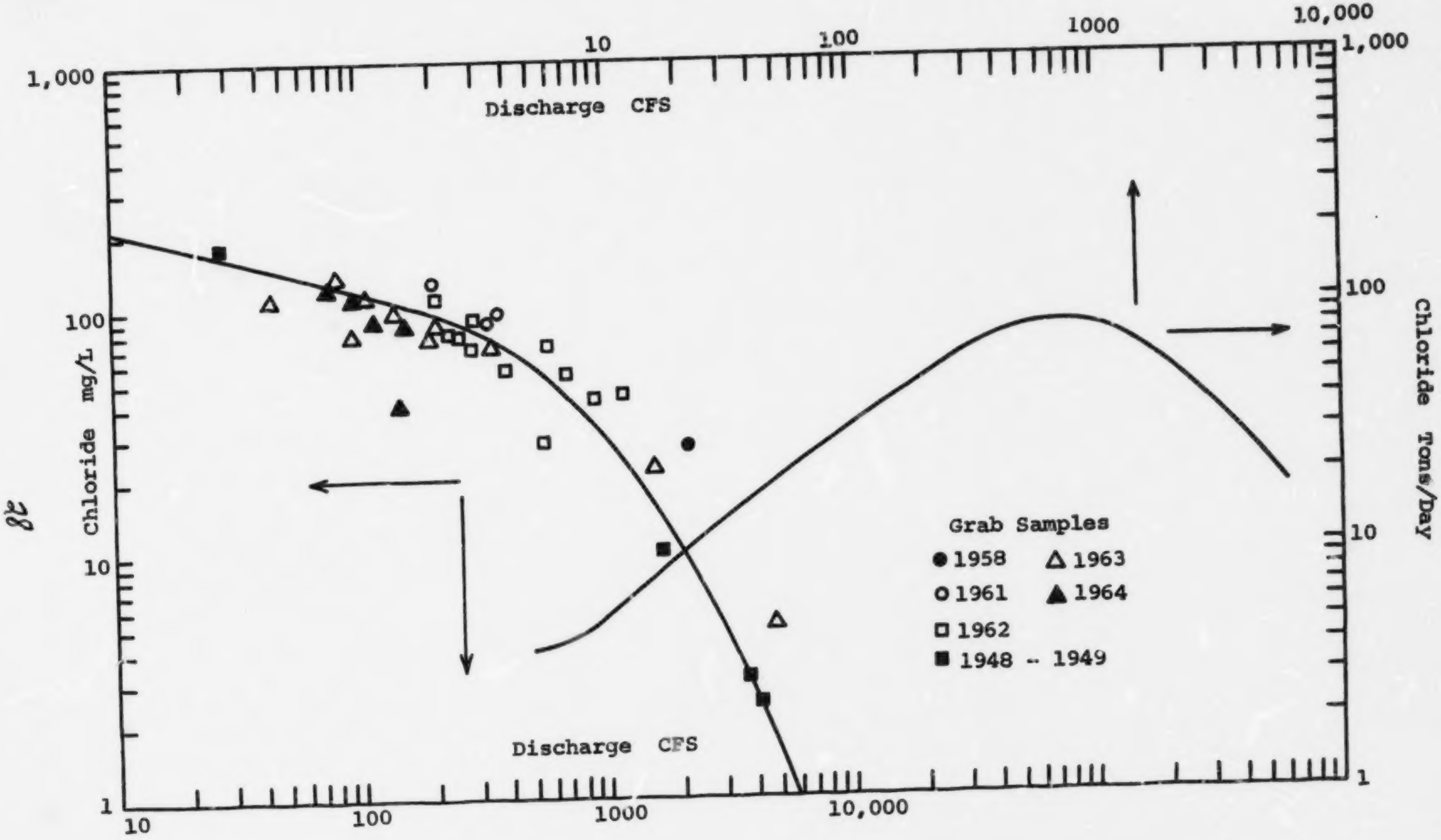


Figure 8 Chloride Concentration and Chloride Load as a Function of Discharge
Quality Station Number 24
Solomon River at Beloit

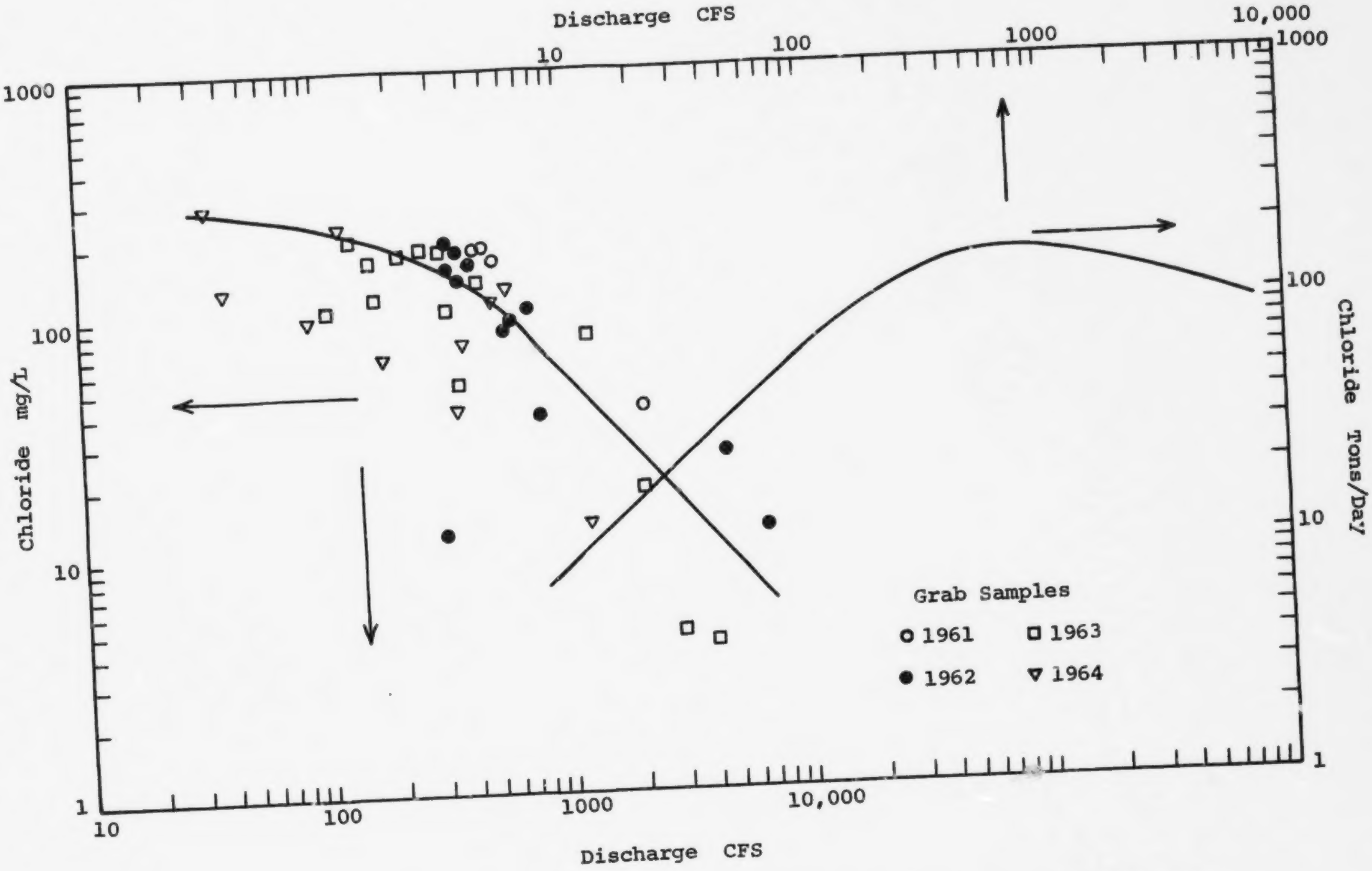


Figure 9 Chloride Concentration and Chloride Load as a Function of Discharge

Quality Station Number 16
Solomon River at Niles

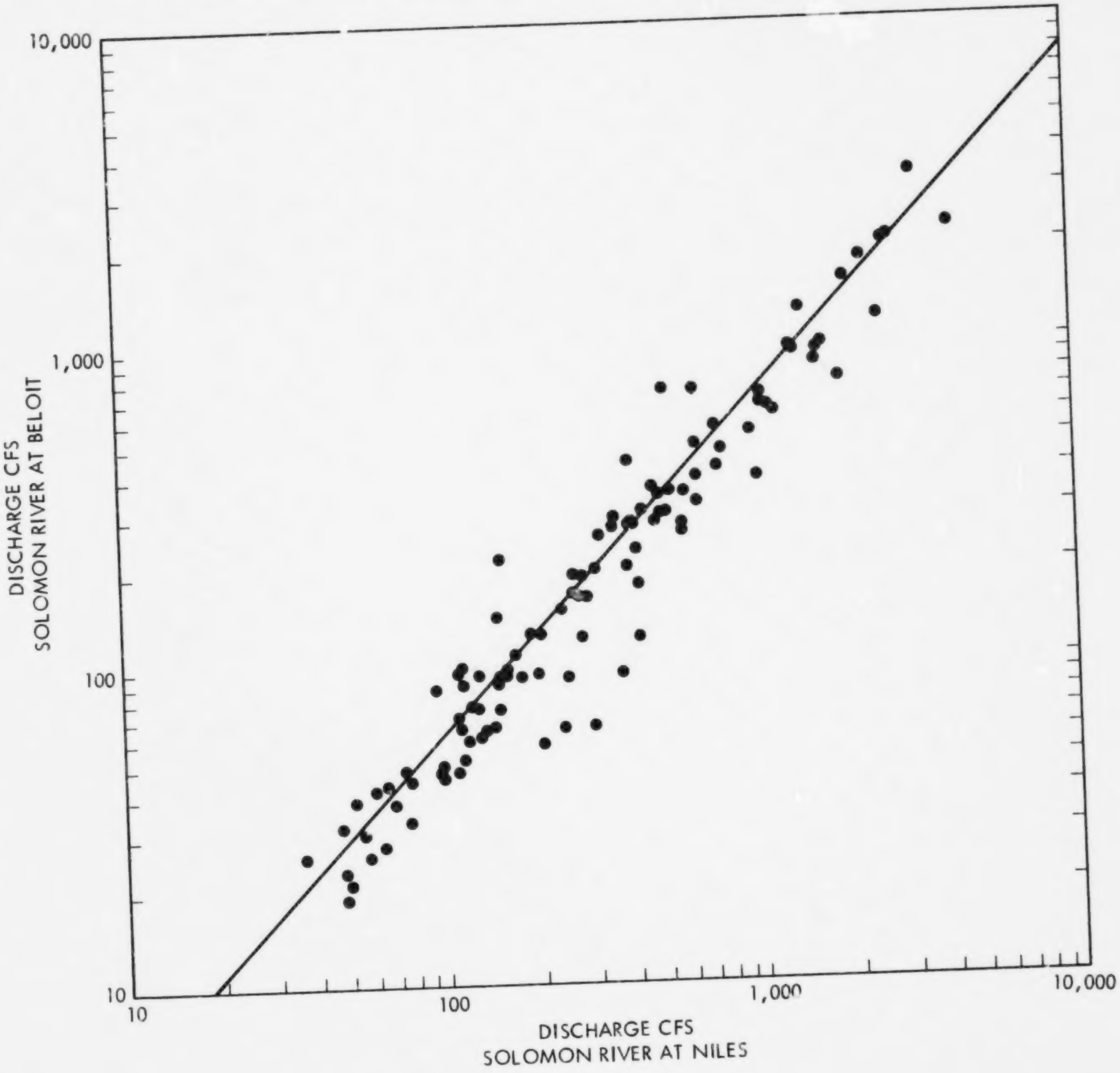


Figure 10 Mean Monthly Discharge Correlation Between the Solomon River at Beloit and the Solomon River at Niles

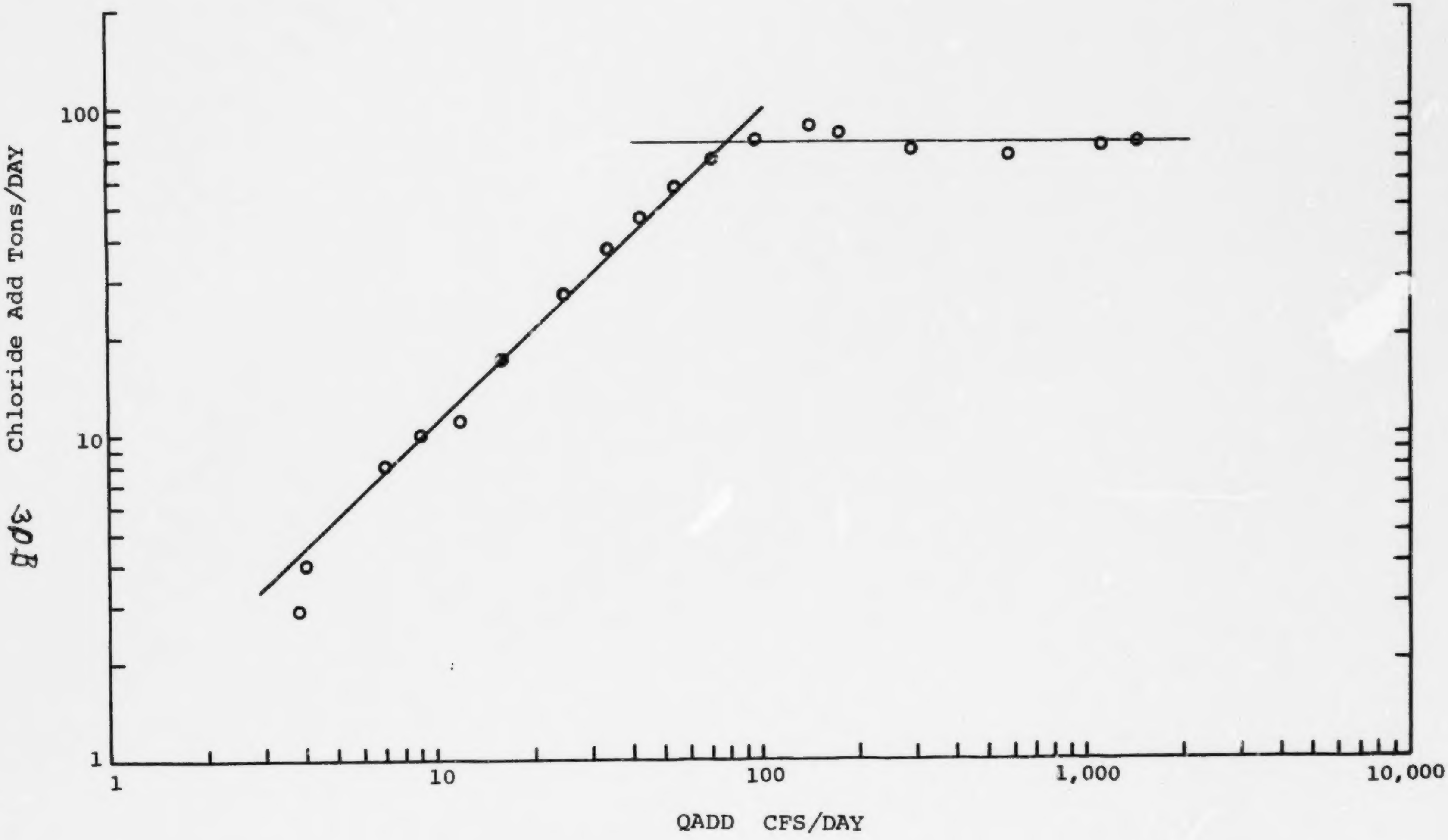


Figure 11 Increase in Chloride Load as a Function of QADD for the Solomon River between Beloit and Niles

last curve is used in the model.

The procedure just described will produce equations which will approximate the concentration of a conservative pollutant anywhere in the river system if the equation of the load-versus-discharge curve is known or can be predicted for each upstream reservoir. The prediction equations for the load increase associated with QADD can be improved significantly in most cases by optimizing them for historic flow periods with the Dynamic Conservative Model discussed in Chapter VI.

Conservative pollutants from either municipal or industrial waste water effluents are independent of the pollution from natural sources. In this case, the pollution load is added at the head of the reach in which it actually occurs and routed through the river system in a manner analogous to the QADD discharge.

At the present time (1966), the only conservative pollution sources in the lower Kansas River basin of potential significance are the DuPont Plant below Topeka, the Cooperative Refinery at Lawrence, and the Sunflower Ordinance Works on Kill Creek near DeSoto.

V. Non-Conservative Water Quality

Although many pollutants can be considered non-conservative, the parameter most commonly associated with this concept is the biochemical oxygen demand (BOD) load imposed on a river system. The prediction of the response of a river to this type of load is difficult, but quite important, because the concentration of dissolved oxygen in the water has many direct effects on water quality.

The most widely-used equations for predicting the response of a river to BOD pollution are modifications of the classical oxygen sag equation proposed by Streeter and Phelps (7). This relationship assumes that the net rate of change in the oxygen deficit of a stream is equal to the difference between two opposing rate equations. These are the removal of oxygen by the BOD exerted on the stream, and the addition of oxygen to the water from the atmosphere. The rate of oxygen removal is approximated by a first-order kinetic equation. The rate of oxygen addition is proportional to the net oxygen deficit. The general equation relating these two reactions takes the form:

$$\frac{dD_t}{dt} = K_1L_t - K_2D_t,$$

where: dD_t/dt = the rate of change of the oxygen deficit with time,

K_1 = deoxygenation rate coefficient (base e),

L_t = ultimate BOD at time (t),

K_2 = reaeration rate coefficient (base e), and

D_t = oxygen saturation deficit at time (t).

When integrated, the equation takes the form:

$$D_t = \frac{K_1L_A}{K_2K_1} [\exp(-K_1t) - \exp(-K_2t)] + D_A\exp(-K_2t),$$

where: L_A = the ultimate oxygen demand at an initial reference point (A) in the stream, and

D_A = the dissolved oxygen saturation deficit at the same point (A).

The basic objections to the classical Streeter-Phelps approach are that it does not account for the following items:

(1) the removal of BOD by sedimentation or adsorption;

(2) the addition of BOD by the scour of bottom deposits or by the diffusion of organic products from the benthal layer into the overlying water;

(3) the addition of BOD along the stretch by local runoff;

(4) the removal of oxygen from the water by diffusion into the benthal layer;

(5) the addition of oxygen by the photosynthetic action of plankton and fixed plants;

(6) the removal of oxygen by the respiration of plankton and fixed plants;

(7) the continuous redistribution of both oxygen and BOD by longitudinal dispersion.

While it is true that the original theory does not specifically account for the above factors, Streeter and Phelps were aware that these factors were acting, and indirectly accounted for many of them by evaluating the rate constants from actual stream data.

A number of modifications to the classical oxygen sag equation are available which take into account one or more of the factors previously mentioned. Among them are the formula suggested by Tsivoglow et al (8), which was subsequently used by Worley (9) on the Willamette River Model. In this case, the equation was modified to include the oxygen demand by sludge deposits and the organic loads from overland drainage. When integrated, it took the form:

$$D_t = \frac{K_1L_A}{K_2-K_1}\ [\exp(-K_1t) - \exp(-K_2t)] + \frac{24S_LV}{K_2}\ [1-\exp(-K_2t)]$$
$$+ \frac{24B_LV}{K_2}\ [\frac{K_2\ [1 - \exp(-K_1t)] + K_1\ [1 - \exp(-K_2t)]}{K_2-K_1}] + D_A\exp(-K_2t),$$

where: D_t, L_A, D_A, K_1, & K_2 are as previously defined,

B_L = Bank load - defined as a uniform oxygen demand (such as agricultural runoff), which may enter the stream along its banks,

S_L = Sludge load - defined as the oxygen demand imposed by the benthal deposits,

V = Velocity of stream flow in miles per hour, and

24 = constant to convert miles per hour to miles per day.

The bank load and sludge load along a reach are evaluated independently and then substituted into the general equation. If these factors are omitted, the equation reduces to the classical formula.

A third sag equation has been developed by O'Connell and Thomas (10) which accounts for the production and consumption of oxygen by benthic algae. It takes the form:

$$D_t = \frac{K_1L_A}{K_2-K_1}[\exp(-K_1t) - \exp(-K_2t)] - \frac{O-R}{K_2}\ [1 - \exp(-K_2t)]$$

$$+ D_A\exp(-K_2t),$$

where: D_K, L_A, D_A, K_1, & K_2 are as previously defined,

O = production of oxygen by photosynthesis in mg/L/day, and

R = oxygen consumed by algal respiration in mg/L/day.

An equation which incorporates all of the individual parameters previously discussed has been independently derived by Camp (11) and by Dobbins (12). In this case, the integrated equation takes the form:

$$D_t = \frac{K_1}{K_2 - (K_1+K_3)} [L_A - \frac{P}{K_1+K_3}] [\exp[-(K_1+K_3) t] - \exp(-K_2 t)]$$

$$+ [\frac{A}{K_2} + \frac{K_1 P}{K_2 (K_1+K_3)}] [1 - \exp(-K_2 t)] + D_A \exp(-K_2 t),$$

where: D_t, L_A, D_A, K_1, & K_2 are as previously defined,

P = rate of addition of BOD along the stretch, (this includes the rate of addition of BOD from the benthal sludge deposit and from the bank load), and

A = rate of removal of dissolved oxygen along the reach. This is the net rate of consumption of oxygen by all processes other than the biological oxidation of the flowing BOD load. The value of P can be positive or negative, depending on the relative magnitude of the various demands and sources of oxygen in the system. The principal components of this factor are:

(1) Oxygen demand of the benthal deposit,

(2) Oxygen used to satisfy the immediate chemical demand of any anaerobic decomposition products emerging from the benthal deposit, and

(3) Net oxygen demand or production by the plankton and fixed plants in the stream.

K_3 = rate constant for BOD removal by sedimentation or adsorption (base e).

The general equation may be simplified if either A, P, or K_3 are equal to zero. If all of these factors are equal to zero, the equation reverts to the classical Streeter-Phelps equation.

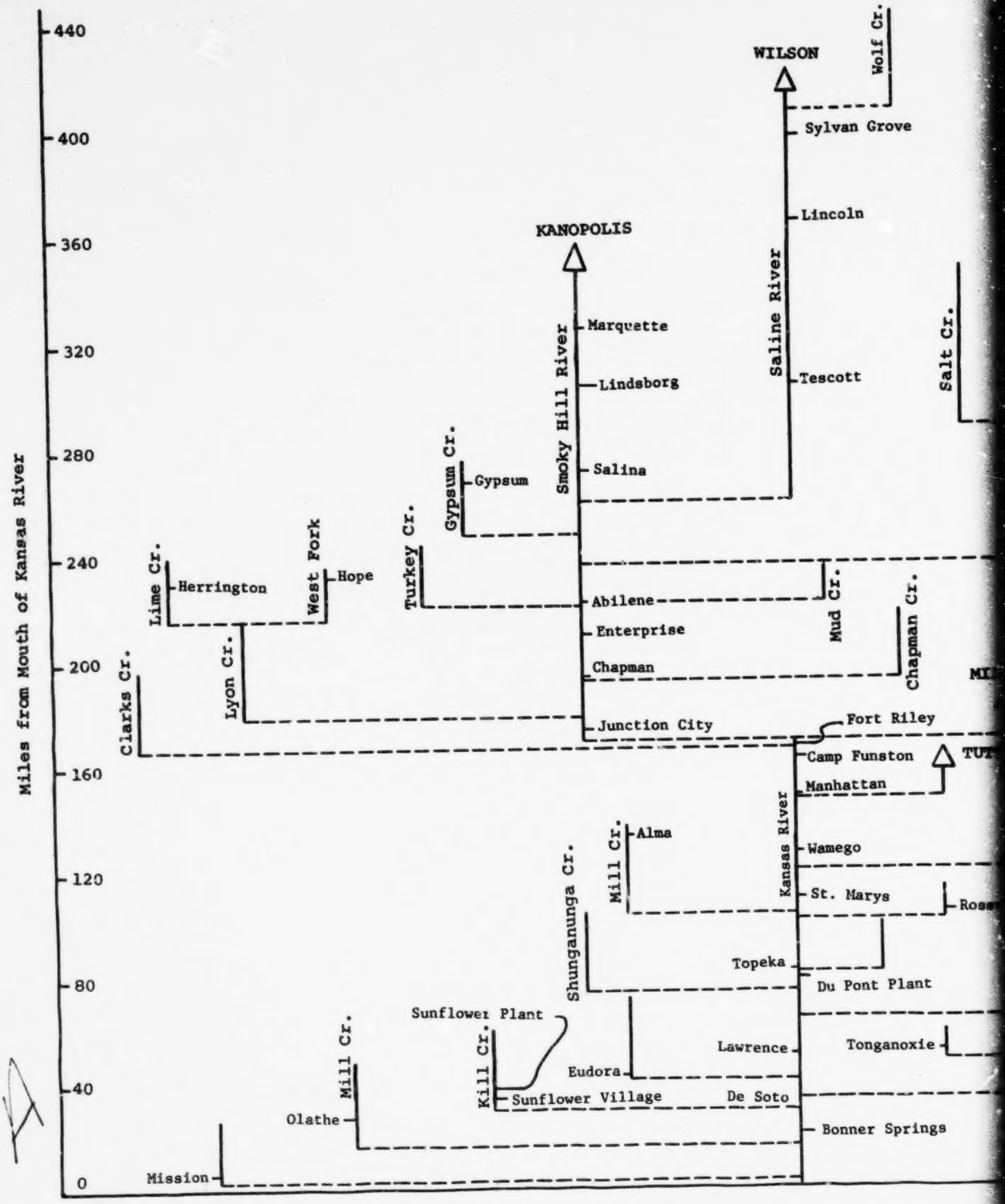
Dobbins (12) has also developed an approach for including the effect of longitudinal dispersion in the dissolved oxygen profile.

The equation developed by Camp and Dobbins has been incorporated into the lower Kansas River model. This relationship was used because it allows the user to determine the sensitivity of the oxygen sag profile to changes in each of the input parameters. The effect of longitudinal dispersion has not been included in the model.

The locations of potential point sources of BOD loading in the lower Kansas River basin are indicated in Figure 12. The river reach designations in the model have been established to permit a BOD input in pounds per day from each potential source. Estimates of the magnitude of current (1966) and future BOD discharge from these sources are available from the Environmental Health Services Division of the Kansas State Department of Health.

The values of many of the other parameters used in the Camp-Dobbins equation are not known for the lower Kansas River basin. One purpose of the field investigations being conducted at the present time is to correct this deficiency. Figures 13 through 15 present a preliminary evaluation of the data collected thus far for summer low-flow conditions. For purposes of this analysis, "summer" was defined to include the months of June, July, and August.

The dissolved oxygen percent saturation values given in Figure 13 were computed by comparing observed values of dissolved oxygen with standard dissolved osygen saturation values (13). The standard dissolved oxygen saturation values used in these computations are given in Table 6.


Miles from Mouth of Kansas River
440
400
360
320
280
240
200
160
120
80
40
0
WILSON
Wolf Cr.
Sylvan Grove
Lincoln
Saline River
Tescott
Salt Cr.
KANOPOLIS
Marquette
Lindsborg
Smoky Hill River
Salina
Gypsum Cr.
Gypsum
Turkey Cr.
West Fork
Hope
Lime Cr.
Herrington
Abilene
Mud Cr.
Enterprise
Chapman
Chapman Cr.
Lyon Cr.
Clarks Cr.
Junction City
Fort Riley
Camp Funston
TU
Manhattan
MI
Kansas River
Alma
Mill Cr.
Wamego
St. Marys
Ros
Shunganunga Cr.
Topeka
Du Pont Plant
Sunflower Plant
Kill Cr.
Eudora
Lawrence
Tonganoxie
Sunflower Village
De Soto
Mill Cr.
Olathe
Bonner Springs
Mission

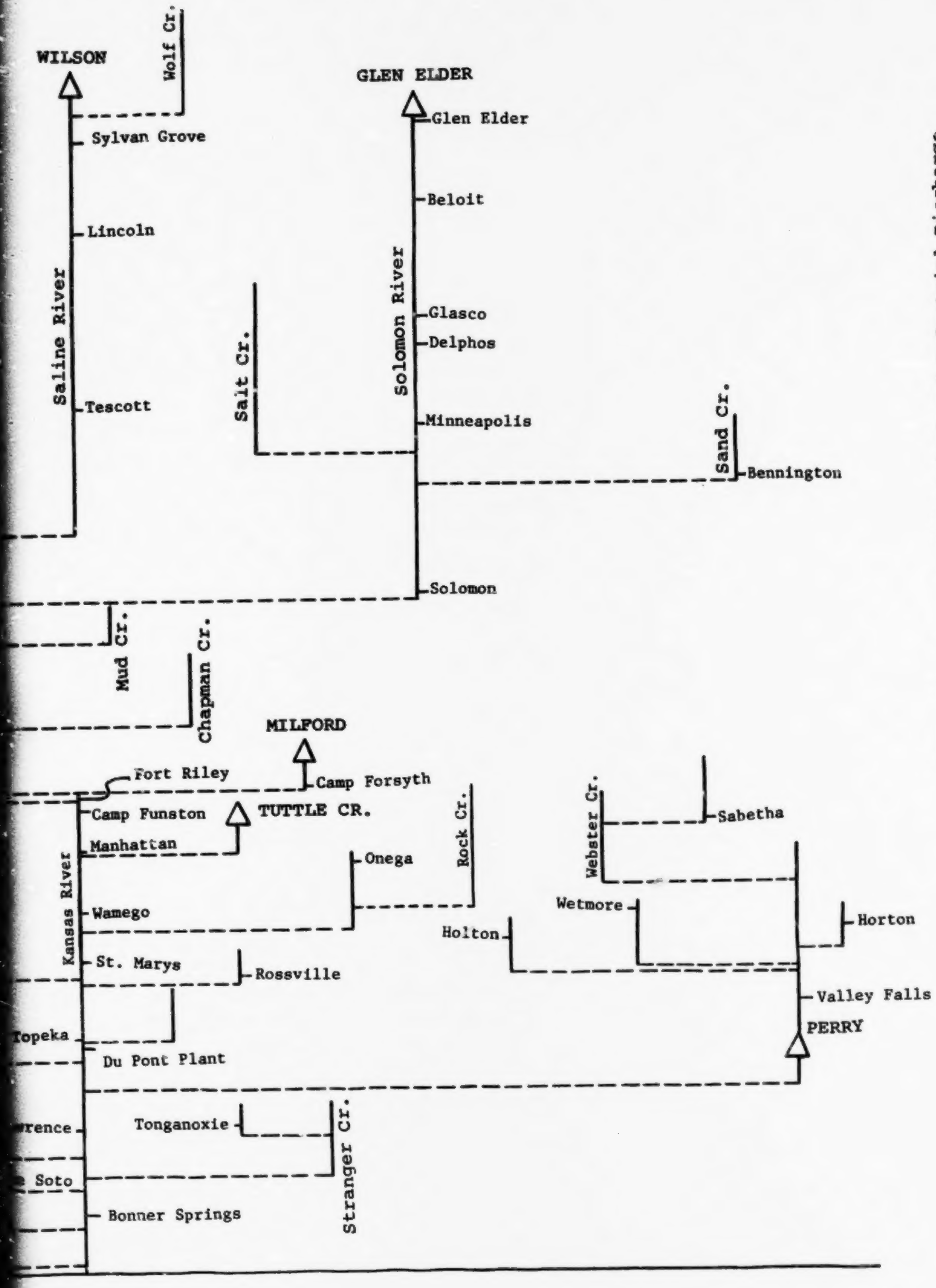

WILSON
Wolf Cr.
Sylvan Grove
Lincoln
Saline River
Tescott
GLEN ELDER
Glen Elder
Beloit
Solomon River
Glasco
Delphos
Salt Cr.
Minneapolis
Sand Cr.
Bennington
Solomon
Mud Cr.
Chapman Cr.
MILFORD
Fort Riley
Camp Forsyth
Camp Funston
TUTTLE CR.
Manhattan
Onega
Rock Cr.
Webster Cr.
Sabetha
Kansas River
Wamego
Wetmore
Horton
Holton
St. Marys
Rossville
Valley Falls
opeka
PERRY
Du Pont Plant
rence
Tonganoxie
Stranger Cr.
e Soto
Bonner Springs

Figure 12 Sources of Municipal and Industrial Discharge in the Lower Kansas River Basin

B

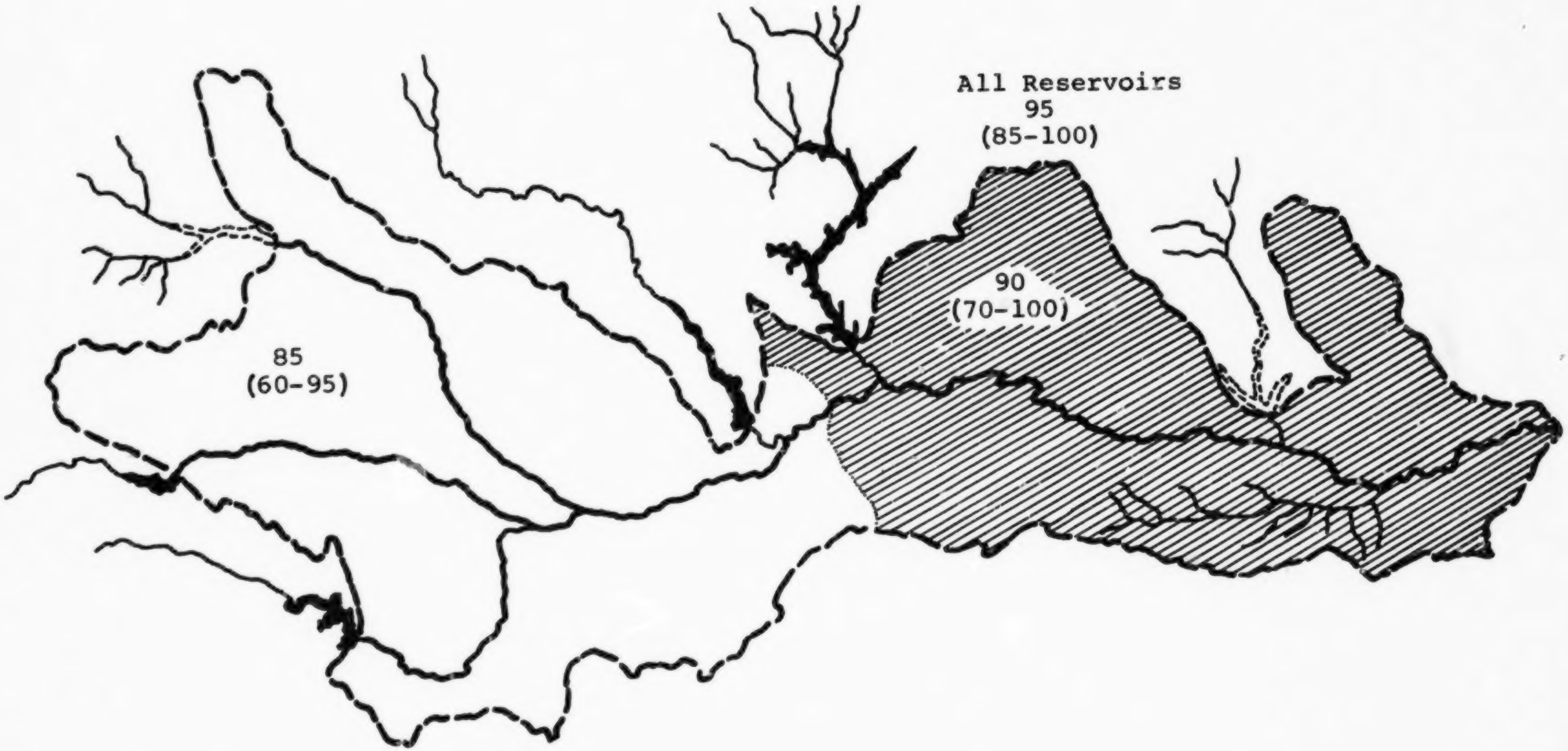

All Reservoirs
95
(85-100)
90
(70-100)
85
(60-95)

Figure 13 Average and Range of the Percent Saturation of Dissolved Oxygen in the Lower Kansas River Basin for Summer Low Flow Conditions

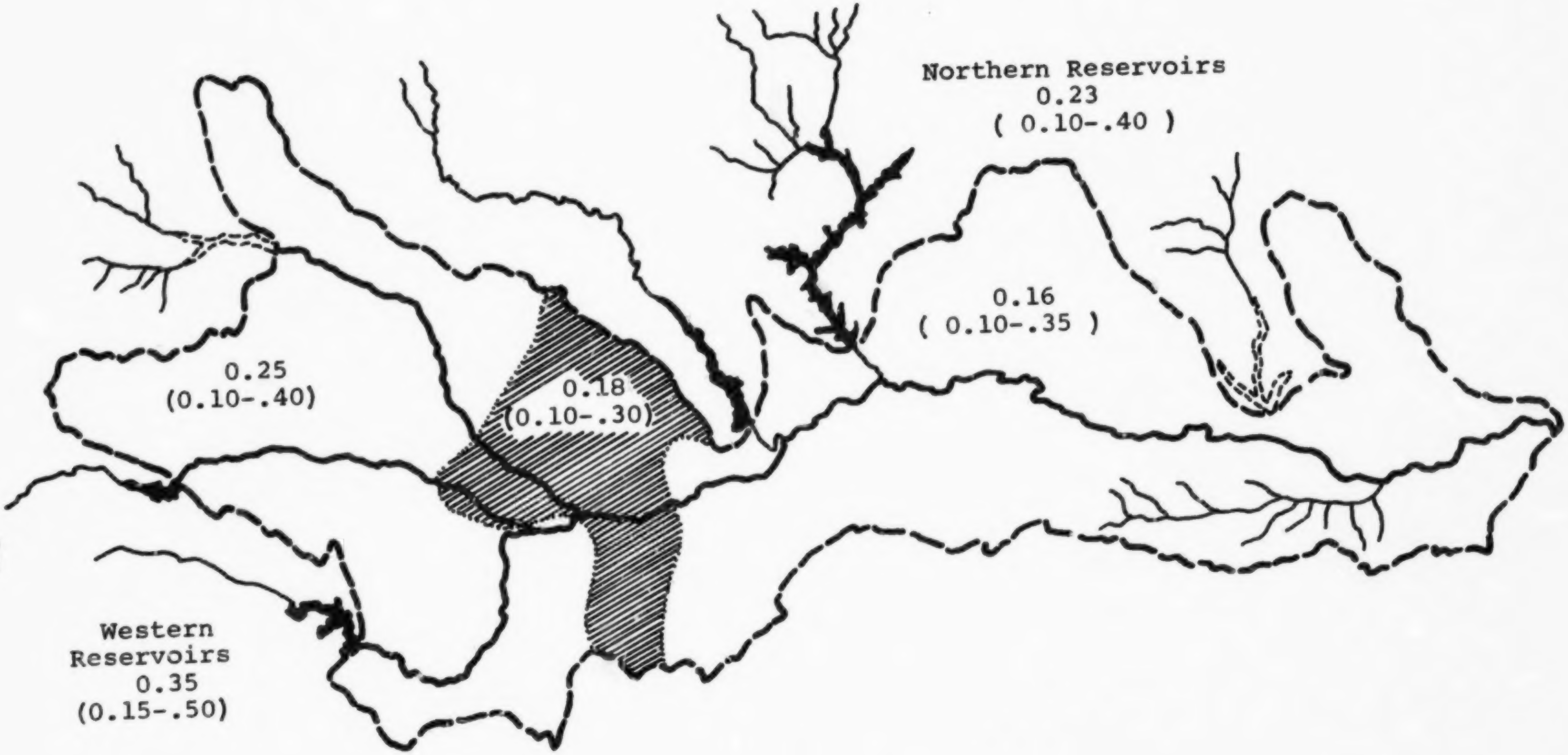

Northern Reservoirs
0.23
(0.10-.40)
0.16
(0.10-.35)
0.25
(0.10-.40)
0.18
(0.10-.30)
Western
Reservoirs
0.35
(0.15-.50)

Figure 14 Average and Range of the Deoxygenation Rate Constant K_1/Day 20°C Base e in the Lower Kansas River Basin for Summer Low Flow Conditions

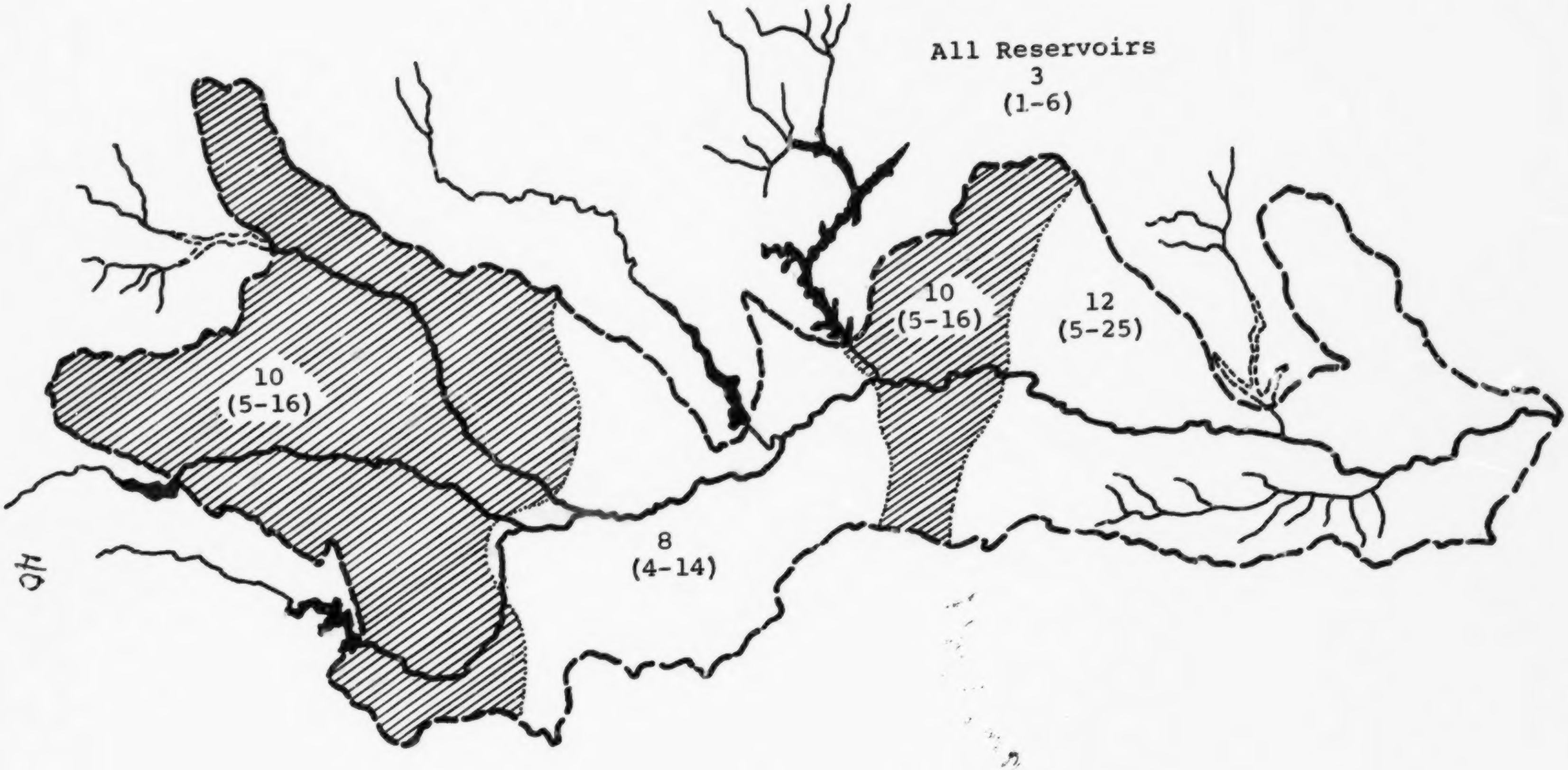

All Reservoirs
3
(1-6)
10
(5-16)
12
(5-25)
8
(4-14)
10
(5-16)

Figure 15 Average and Range of the Ultimate Biochemical Oxygen Demand in the Lower Kansas River Basin for Summer Low Flow Conditions

Table 6

Dissolved Oxygen Saturation Values for the Lower Kansas River Basin Summer Conditions

Temperature °C	Dissolved Oxygen mg/L	Temperature °C	Dissolved Oxygen mg/L
20	8.7	28	7.4
21	8.5	29	7.3
22	8.3	30	7.1
23	8.2	31	7.0
24	8.0	32	6.8
25	7.9	33	6.7
26	7.7	34	6.6
27	7.6		

These saturation values have been adjusted to compensate for the change in barometric pressure occurring because of ground elevation and atmospheric temperature by assuming an average summer pressure of 731 millimeters of mercury throughout the lower Kansas River basin.

Dissolved oxygen supersaturation was observed occasionally at a number of stations throughout the basin. If these values were greater than 110 percent of saturation, they were excluded from the computations from which Figure 13 was derived.

The light-bottle biochemical oxygen demand measurements conducted by the Kansas State Department of Health also indicated algal activity at a number of stations throughout the basin. At the present time, this activity is quite sporadic and, including the oxygen resource contributed by photosynthetic action, does not appear warranted. This condition may be altered after all of the upstream reservoirs are in operation and the suspended sediment load in the lower basin is reduced.

The average and range of the deoxygenation rate constants given in Figure 14 and the ultimate oxygen demand values in Figure 15 were computed from three- and six-day BOD observations by the method suggested by Rhame (14). This procedure provided an acceptable method for computing these values with a minimum of laboratory data.

The BOD samples were analyzed by the Kansas State Department of Health. The procedures set forth in Standard Methods (15) were used in conducting these tests. No nitrification suppression techniques were employed. Apparent nitrification was observed occasionally at several stations throughout the basin. These observations were excluded from the computations used in obtaining Figures 14 and 15.

It should be emphasized that most of the water quality stations used in the 1966 summer survey are located upstream of any immediately adjacent municipal or industrial waste-water outfalls. The only exceptions to this rule are the Kansas River at Kansas City, the Kansas River at Bonner Springs, and the Saline River at Tescott. The municipal pollution load at Bonner Springs has little effect on the water quality in the Kansas River at the sampling station.

The values of the water quality parameters given in Figures 13 through 15, therefore, depict the non-conservative quality of the water within the basin before the addition of municipal or industrial waste-water sources. The major source of the biodegradable organic material indicated in Figures 13 through 15 is runoff from agricultural land. The concentration of organic material in this runoff is significant. However, as indicated by the relatively low deoxygenation rate constants, the material is stabilized relatively slowly.

The effect of organic pollution from overland flow has been included in the mathematical model by arbitrarily establishing a logarithmic relationship between BOD load and QADD discharge. This relationship is illustrated in Figure 16, and is assumed to apply throughout the lower Kansas River basin.

The equation given in Figure 16 is undoubtedly a gross simplification of the relationship between BOD load and

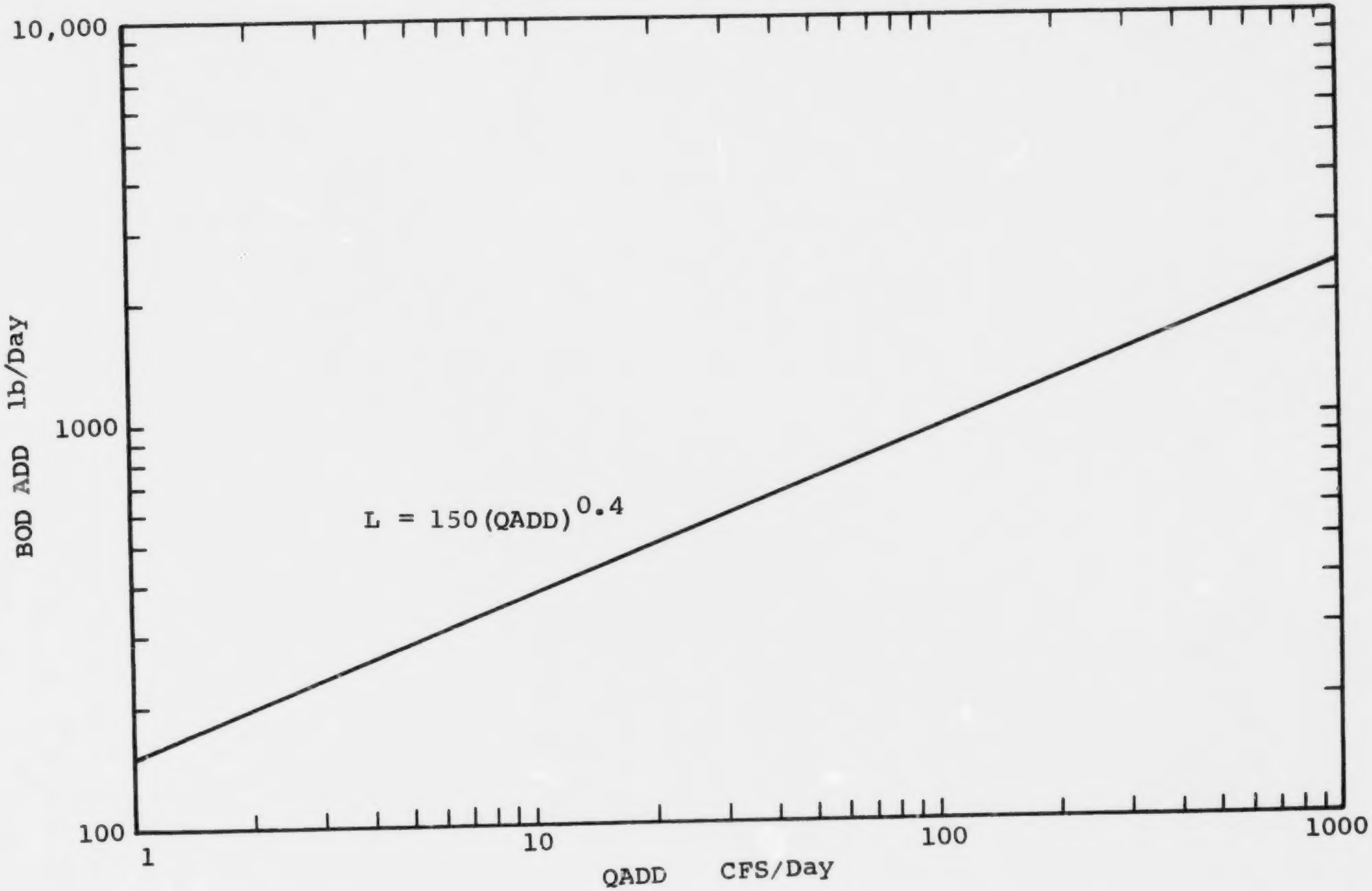


Figure 16 BOD Loading as a Function of QADD for the Lower Kansas River Basin Model

agricultural runoff. There is probably a seasonal and an antecedent rainfall effect which is not included in the present approach. The equation does, however, produce BOD concentrations which appear reasonable in light of the work being conducted at the Robert A. Taft Sanitary Engineering Center at the present time (16).

Under certain conditions, observed BOD and dissolved oxygen deficit values can be used to evaluate the reaeration rate constant (K_2). However, because the reaeration rate constant is a parameter which is physically independent of BOD and is a function only of the dissolved oxygen deficit and the turbulence of the stream, a number of equations have been proposed in which it is related directly to the physical geometry of the stream.

Among these approaches is that proposed by O'Connor and Dobbins (17) which took the form:

Case 1. Non-Isotropic Turbulence (shallow streams)

$$k_2 = 480S^{0.25}D_L^{0.5}H^{-1.25}$$

where: k_2 = reaeration rate constant/day (base 10),
D_L = coefficient of molecular diffusion of oxygen in water ft^2/day,
D_L = 0.00207 ft^2/day at 20°C, and can be estimated for other temperatures by the equation

$$D_{L(T)} = D_{L(20)}(1.037)^{(T-20)},$$

S = slope of river channel in ft/foot, and
H = average water depth ft.

Case 2. Isotropic Turbulence

$$k_2 = 0.434D_L^{0.5}V^{0.5}H^{-1.5},$$

where: D_L and H are as previously defined, and
V = stream velocity ft/sec.

The Chezy coefficient was used to distinguish between isotropic and non-isotropic turbulence:

C greater than 17--isotropic,
C 20 to 14---------transition zone,
C less than 17-----non-isotropic.

In a subsequent paper O'Connor (18) pointed out that the isotropic equation provided a better fit to the experimental data, and suggested that the non-isotropic equation be discarded. The O'Connor-Dobbins equations were used in the Willamette Model Study (9).

Krenkel and Orlob (19) have related the reaeration coefficient to turbulence as described by the eddy-diffusion analogy. Their investigations lead to the development of a multiple correlation equation of the form:

$$k_{2(20)} = (1.138 \times 10^{-5}) K_d^{1.321} H^{-2.32},$$

multiple correlation coefficient = 0.963,

where: k_d = the effective eddy diffusion coefficient (ft^2/min).

A modification of this equation was used in the Sacramento-San Joaquin Water Quality Model (20). In this case, the effect of temperature was ignored, and the equation was simplified to:

$$K_2 = 2.5 \frac{V}{H},$$

where: H & V are as previously defined, and
K_2 = reaeration rate constant/day (base e).

Churchill et al (21) have conducted one of the few extensive field evaluations of the reaeration coefficient. Based on their work, the parameter may be evaluated by:

$$k_{2(20)} = 5.026 V^{0.969} H^{-1.673},$$

multiple correlation coefficient = 0.822.

The range of observations used in developing the equation were:

V = 2.10 to 5.00 ft/sec. H = 2.12 to 11.41 ft.

Owens et al (22) have extended the range of the previous equation by combining data taken by them in English streams with the data previously obtained by Churchill and Gameson. The regression equation produced by this mass of data is:

$$k_{2(20)} = 9.41V^{0.67}H^{-1.673},$$

multiple correlation coefficient = 0.95.

The range of observations used in developing this equation were:

$V = 0.1$ to 5.0 ft/sec. $H = 0.4$ to 11.0 ft.

An equation for predicting the reaeration rate for slow flowing, shallow streams was developed by segregating the data. In this case:

$$k_{2(20)} = 10.1\ V^{0.73}H^{-1.75},$$

multiple correlation coefficient = 0.94.

The range of observations used in developing this equation were:

$V = 0.1$ to 5.0 ft/sec. $H = 0.4$ to 11.0 ft.

In developing these equations, Owens used the oxygen saturation values suggested by Truesdale et al (23). These values are now considered to be significantly lower than the accepted standards.

Like the deoxygenation coefficient, the reaeration coefficient is temperature-dependent. Considerable confusion exists in the literature regarding the exact nature of this dependence. The standard value is now considered to be (13):

$$k_{2(T)} = k_{2(20)}(1.0241)^{(T-20)}.$$

This correction should be applied to the reaeration coefficient calculated by the Streeter-Phelps, the Churchill, and the Owens technique. Presumably, it may also be applied to the O'Connor-Dobbins technique, if the coefficient of molecular diffusion (D_L) is considered a constant. Temperature corrections have not been applied to the Krenkel-Orlob approach in practice.

Dobbins (12) has also introduced a formulation for the reaeration coefficient based on a theoretical model in which the interfacial liquid film is maintained in only the statistical sense. Under this concept the equation takes the form:

$$k_2 = \frac{0.12C_A AE^{0.375} \coth(BE^{0.125}C_4^{-0.5})}{C_4^{1.5}H},$$

where: $C_A = 1.0 + F^2$,

$C_4 = 0.9 + F$,

$A = 0.68 + 0.054(T-20)$,

$B = 0.976 \div 0.0137(30-T)^{1.5}$,

$E = 30SV$,

S = slope ft/100 ft.,

$F = Vg^{-0.5}H^{-0.5}$, and

G = acceleration of gravity.

H, V, & T are as previously defined.

In this theory the change in the reaeration coefficient due to temperature is not considered constant. The temperature effect is, however, automatically built into the equation.

In order to evaluate the applicability of the reaeration equations available in the literature, a comparative study between the O'Connor-Dobbins isotropic equation (18), the equation developed by Churchill et al (21), the two equations proposed by Owens et al. (22), and the recent equation developed by Dobbins (12) was conducted for a number of stations in the lower Kansas River basin.

The velocity and hydraulic depth values used in the study were obtained from the logarithmic relationships described in Chapter III. The bed slope values used in the Dobbins equation were calculated from the discharge rating curves for the gaging stations operated in the basin by the United States Geological Survey and from the river profile of the Kansas River developed by the United States Corps of Engineers. The results shown in Figure 17 are typical of the results obtained on all of the stations located on the main stem of the Kansas River. Larger deviations were common in the smaller rivers in the basin.

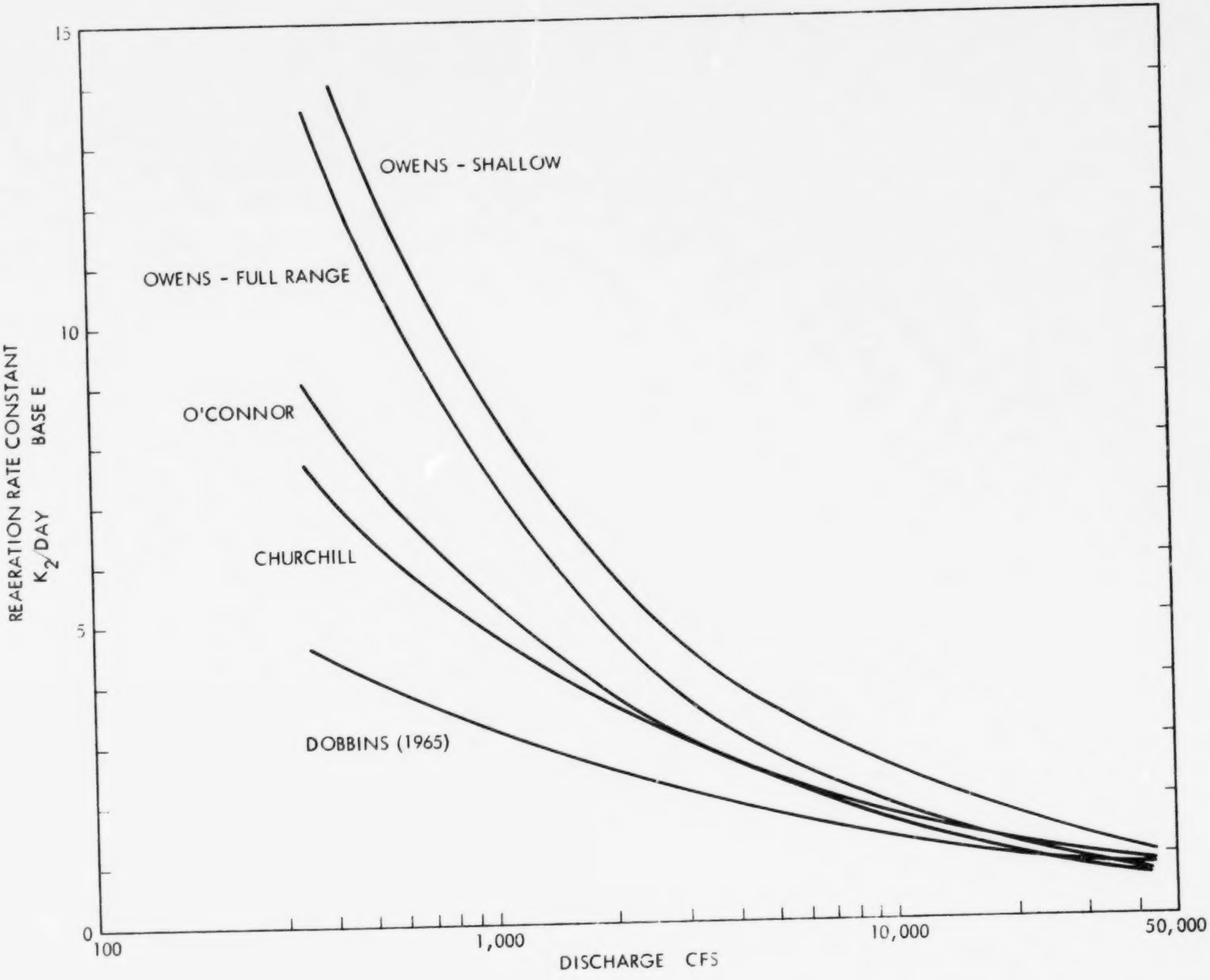


Figure 17 Computed Reaeration Rate Constants as a Function of Discharge for the Kansas River at Topeka

A field evaluation of the reaeration rate constant at selected points throughout the basin is being conducted by the Kansas State Department of Health at the present time. Until the results of this study are available, the equation developed by Dobbins (12) has been arbitrarily incorporated into the mathematical model.

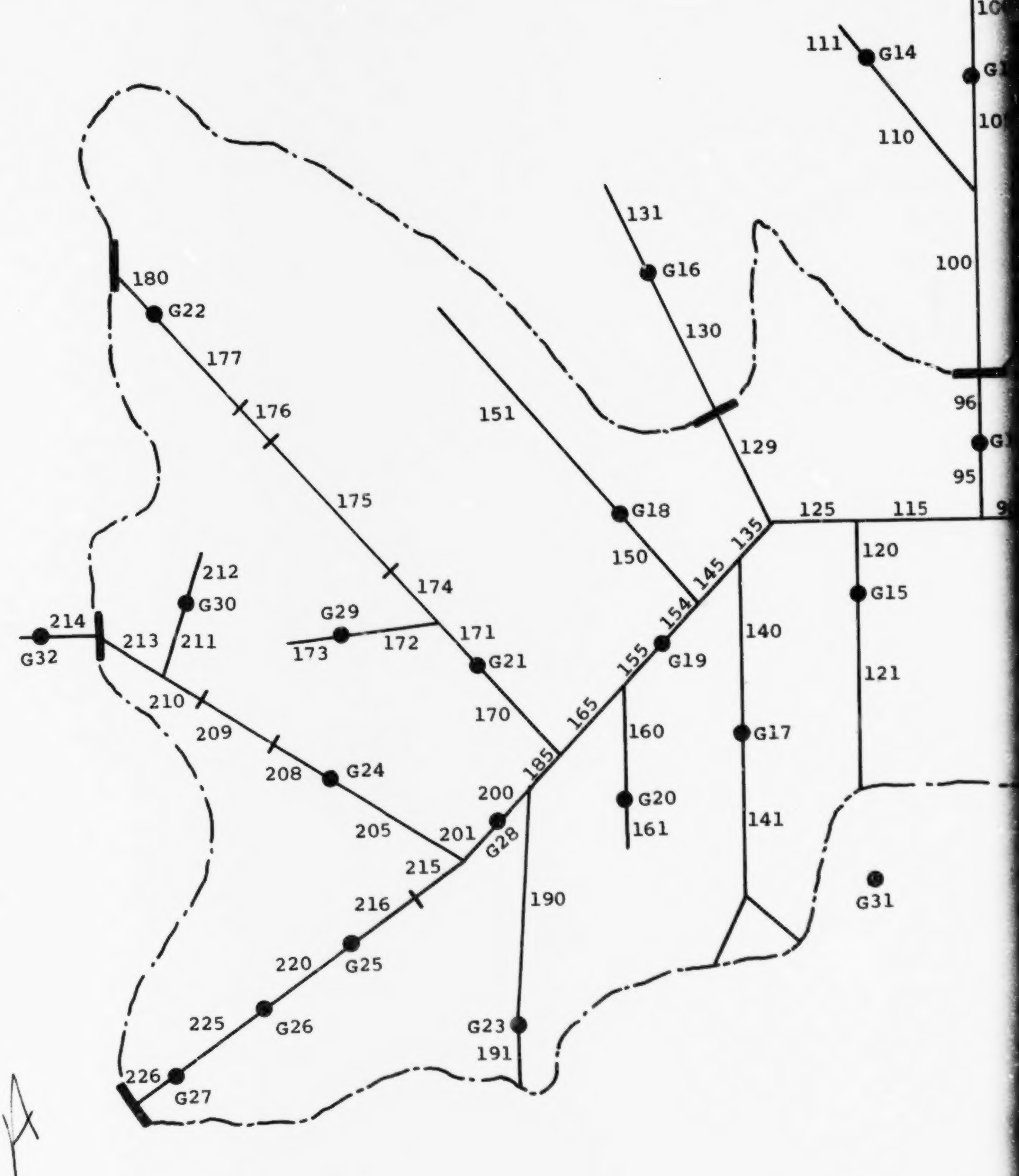
VI. The Mathematical Model

The modeling of the water quality of a river system may be approached in a variety of ways. The simulation of the prototype may be achieved by physical analogy or by computer solutions of the mathematical models of the physical and chemical processes observed in the prototype. This study was restricted to a consideration of mathematical models suitable for a large-capacity, high-speed, digital computer.

The network of streams in the basin was subdivided into 30 reaches, as shown in Figure 18. Reach boundaries were selected such that hydraulic conditions within the reach were approximately constant. New reaches were started below tributary junctions and below any known point source of pollution. The reach identification numbers were chosen such that a given reach could be subdivided into several parts without disrupting the identification of other reaches.

A natural river basin produces a highly unsteady flow system in response to a highly variable hydrolic input. Any attempt to simulate such a system requires the imposition of certain simplifying restrictions. The randomness of the natural system often is replaced by an analysis of selected extreme conditions. The unsteady nature of the dynamic system is often replaced with a steady-state or quasi-steady-state system. Both of these simplifications depart from reality. For some purposes, such departures do not seriously distort the desired solutions.

Models designed to simulate the behavior of a system of proposed structures over a long time period cannot ignore the random nature of basin hydrology. Traditional methods relied on an analysis of the short historical record for the basin. Proposed structures were evaluated on their response to a repeat of the histroical events. Recent studies in operational hydrology have permitted the application of statistics to evaluate these structures on their response to series of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G18
125
115
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120
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174
G15
G30
154
G29
214
171
140
213
211
172
155
G19
G32
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121
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170
160
G17
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185
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G24
200
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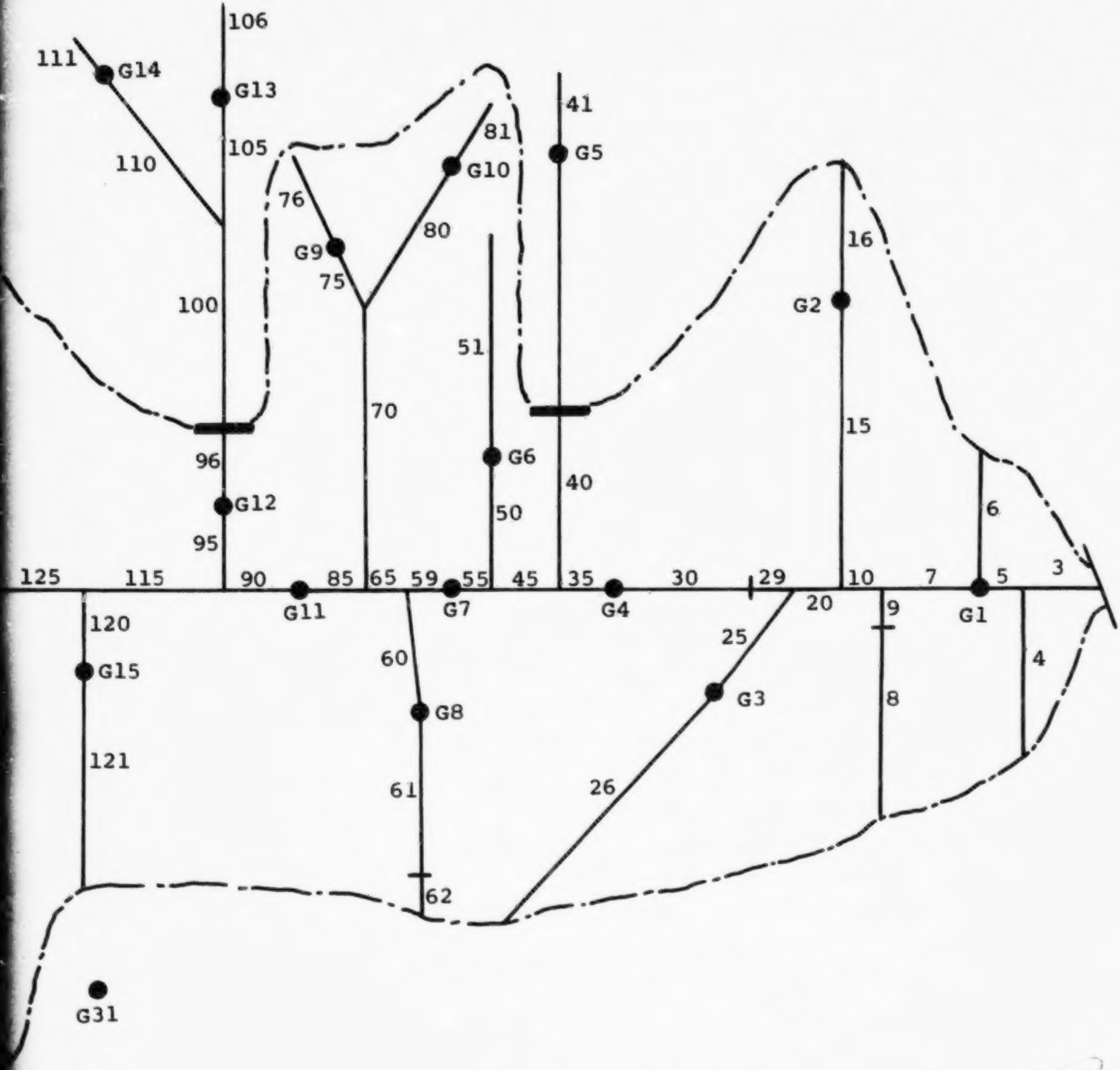
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G7
55
45
35
G4
30
29
20
10
9
7
G1
5
3
120
G15
60
G8
25
G3
4
8
121
61
26
62
G31

B

Figure 18 Reach and Gage Numbering System for the Lower Kansas River Model

events which could probably occur during the economic life of the project.

Models designed to simulate the response of a river system to a specific change in operating conditions are not concerned with random events. Indeed, such a simulation relies on deterministic solutions to differentiate between the system response with and without the proposed change in operating conditions. The model described in this report is of the deterministic type. In this case, the objective was to determine the effect on water quality throughout the system due to proposed changes in pollution loadings at given locations. Another objective was to determine the best operating procedure for the reservoirs.

The choice between dynamic and quasi-steady-state models is dependent on the use for which the model is constructed. Considering the regulatory and operational nature of the desired output, it was decided initially to build a quasi-steady-state model. In this way the degradation of non-conservative pollutants would be time-dependent, but the hydrology of the system would be assumed steady. This is a basic assumption of the Streeter-Phelps oxygen-sag relationships. It was assumed that the boundary conditions would be established arbitrarily so as to investigate a specific set of conditions in the basin. A great simplification of the program is achieved if it can be assumed that the flows past the tributary gages are single-valued. This assumption is clearly in violation of the natural basin, but permits the investigation of the overall effect on basin water quality caused by specific changes in the boundary conditions.

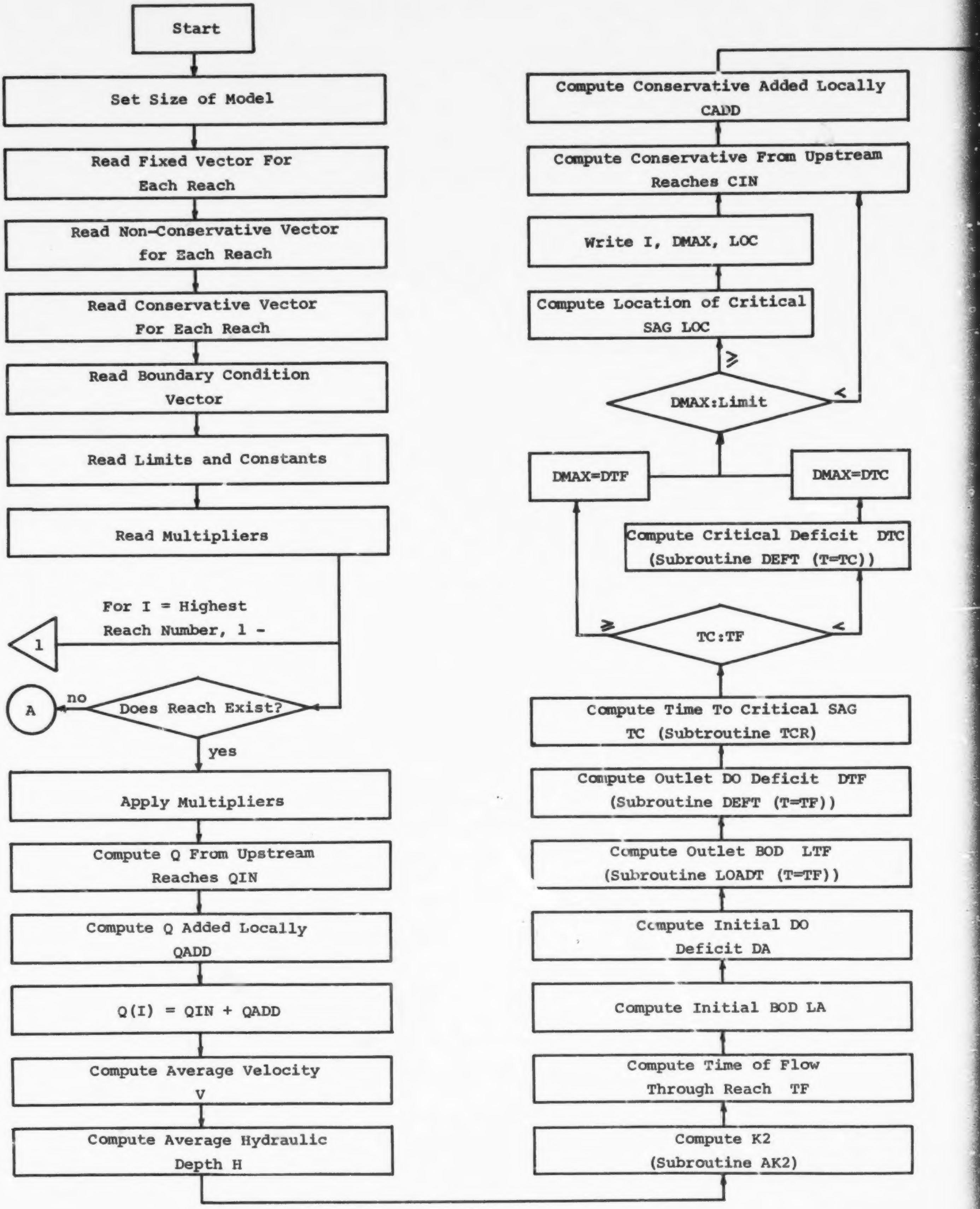
Verification of the ability of the quasi-steady-state model to simulate the natural river system required the use of historic stream flows for the boundary-condition vector. Such stream flows do not behave according to the assumptions of the quasi-steady-state model. The time of flow through the entire model is on the order of fifteen to twenty days. During such a time period, the flow in a given tributary will vary over a considerable range. The correct addition of flows meeting at a junction point requires that the stream gages be correctly lagged in time, to allow for the time of travel of the flow variation through the tributary. The "dynamic model" was developed to meet this need.

The final result of this project was the development of three separate models. The first computer program, called the "Quasi-Steady-State Model," considers both conservative and non-conservative pollutants. The second and third models are similar to each other in that they are dynamic models. The computer storage required to handle hydrographs instead of single values prevented the inclusion of both conservative and non-conservative pollutants in a single dynamic model.

The second computer program, called the "Dynamic Non-Conservative Model," considers only one form of degradable waste. In its present form, this model is set up to handle the carbonaceous biochemical oxygen demand of organic pollution. This model could be used for any decay-type pollution, such as nuclear waste or thermal pollution.

The third computer program, called the "Dynamic Conservative Model," will handle conservative pollutants. Because of computer storage limitations, it is possible to analyze only one pollutant at a time.

The flow diagrams for all three models are shown in Figures 19, 20, and 21. Following each flow diagram is a detailed description of each computer operation.


Start
Set Size of Model
Read Fixed Vector For Each Reach
Read Non-Conservative Vector for Each Reach
Read Conservative Vector For Each Reach
Read Boundary Condition Vector
Read Limits and Constants
Read Multipliers
For I = Highest Reach Number, 1 -
1
A
no
Does Reach Exist?
yes
Apply Multipliers
Compute Q From Upstream Reaches QIN
Compute Q Added Locally QADD
Q(I) = QIN + QADD
Compute Average Velocity V
Compute Average Hydraulic Depth H
Compute K2 (Subroutine AK2)
Compute Time of Flow Through Reach TF
Compute Initial BOD LA
Compute Initial DO Deficit DA
Compute Outlet BOD LTF (Subroutine LOADT (T=TF))
Compute Outlet DO Deficit DTF (Subroutine DEFT (T=TF))
Compute Time To Critical SAG TC (Subtroutine TCR)
TC:TF
≥
<
Compute Critical Deficit DTC (Subroutine DEFT (T=TC))
DMAX=DTF
DMAX=DTC
DMAX:Limit
≥
<
Compute Location of Critical SAG LOC
Write I, DMAX, LOC
Compute Conservative From Upstream Reaches CIN
Compute Conservative Added Locally CADD

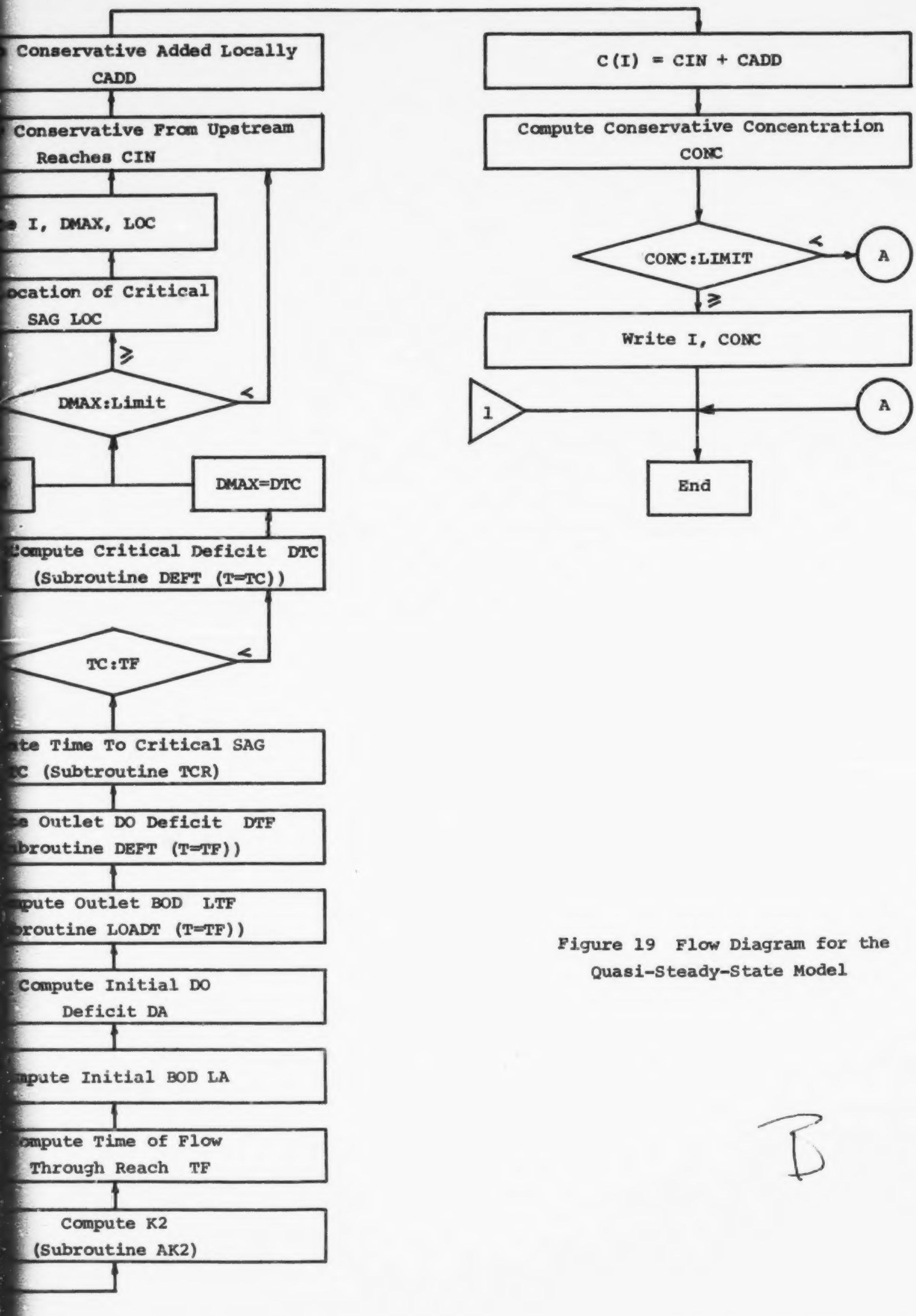

Conservative Added Locally CADD
Conservative From Upstream Reaches CIN
e I, DMAX, LOC
ocation of Critical SAG LOC
≥
DMAX:Limit
<
DMAX=DTC
Compute Critical Deficit DTC (Subroutine DEFT (T=TC))
TC:TF
<
te Time To Critical SAG
C (Subtroutine TCR)
e Outlet DO Deficit DTF
broutine DEFT (T=TF))
pute Outlet BOD LTF
routine LOADT (T=TF))
Compute Initial DO Deficit DA
pute Initial BOD LA
ompute Time of Flow Through Reach TF
Compute K2 (Subroutine AK2)
C(I) = CIN + CADD
Compute Conservative Concentration CONC
CONC:LIMIT
<
A
≥
Write I, CONC
1
A
End

Figure 19 Flow Diagram for the Quasi-Steady-State Model

B

Description of Computer Operations

Quasi-Steady-State Model

1. Set Size of Model

The size of the river system model is established by the number of reach designations and the number of stream gages in the system. These numbers are read in at the beginning to reserve the required amount of storage in the computer.

2. Read Fixed Vector for Each Reach

The fixed physical character of the river system is established by a set of numbers giving the following information for each reach.

UR1 - identification of an upstream reach.

UR2 - identification of a second upstream reach. Except for a reach downstream from a junction, this number will call for zero inputs.

L - length of the reach in river miles.

HA - coefficient for depth vs. discharge function.

HB - exponent for depth vs. discharge function.

VA - coefficient for velocity vs. discharge function.

VB - exponent for velocity vs. discharge function.

QAA - coefficient for QADD function.

QAB - constant for QADD function.

IG1 - identification of a local stream gage for QADD function.

IG2 - identification of a second local stream gage for QADD function.

S - mean bed slope of the reach.

3. Read Non-Conservative Vector for Each Reach

The amount and nature of the organic load entering each reach is given by the following.

ORGA - BOD of point source pollution, such as a sewer outfall. This value is the average daily load as supplied by the State or Federal Regulatory Agency.

K1 - Deoxygenation rate coefficient (base e).

K3 - Rate constant for BOD removal by sedimentation or absorption (base e).

A - Rate of removal of dissolved oxygen along the reach.

P - Rate of addition of BOD along the reach due to the existence of sludge banks. The wash load customarily included in this term is handled separately in this model by applying a wash-load vs. QADD function to the predicted QADD, and treating this load as a point load at the beginning of the reach.

4. Read Conservative Vector for Each Reach

The conservative pollutant is assumed to enter the stream as a function of the local inflow QADD. Field data indicate a non-linear relationship when plotted on log-log paper. This non-linear trend is approximated by several straight line segments. Each segment can be represented by an equation of the form:

$$\text{CADD} = a\ \text{QADD}^b$$

The breakpoint between adjacent segments can be identified by the value of QADD at that point. The functional relationship can be defined by a set of numbers representing the coefficient and exponent for each segment and the value of QADD at the breakpoints. The program will accept up to three segments by means of the following identifiers.

CAA - coefficient of first segment.

CAB - exponent of first segment.

CAC - coefficient of second segment.

CAD - exponent of second segment.

CAE - coefficient of third segment.

CAF - exponent of third segment.

QADC - value of QADD at end of first segment.

QADC2 - value of QADD at end of second segment.

Such a set of identifiers is read in for each reach.

The conservative load entering the system from a reservoir presents a special problem. In this case the pollution load is a function of the discharge from the reservoir and is quite independent of local runoff. The load-discharge relationships for each reservoir have been determined from field data, and the log-log plotted curves are approximated by a number of straight line segments. The program will accept up to eleven segments for a given function.

5. Read Boundary Condition Vector

The Quasi-Steady-State model is constructed on the premise that the flows at a set of tributary stream-flow gages constitute the boundary conditions for the system. The flow at each gage is assumed to be constant, so that there is no need to lag one gage with respect to another. This does not prevent the determination of the time history of a shock load as it travels through the model. The artificial control of flow in some tributaries by reservoir operation becomes part of the boundary-condition vector as this control is reflected in the assigned value for the stream gage below the reservoir. The values for the boundary-condition vector can be assigned arbitrarily or can be set to simulate a natural condition.

6. Read Limits and Constants

The decision as to whether or not a given level of water quality is satisfactory must be based on some established limiting values of measurable quality parameters. These limiting values are read into the computer so that there can be an internal evaluation of the acceptability of water quality for each reach. Reach data are printed out only when the chosen limits have been exceeded.

Some parameters are constant for all reaches. These constants, such as the D.O. concentration at saturation for the given water temperature, are read in only once for a given run.

7. Read Multipliers

In a model of this type it is desirable to be able to change the magnitude of a set of variables by the application of a single multiplier. For example, all municipal loads can be doubled to observe the effect on the system of an overall increase in population.

8. Iteration Through All Reaches

The reaches are numbered in ascending order from the mouth. In actual simulation, however, the program starts at the uppermost or highest-numbered reach and proceeds in the direction of flow. At each junction point the reach numbering transfers the simulation to the uppermost reach of the tributary.

The integers actually assigned to the reaches proceed in steps of five, so as to leave space for future revision of some reaches. In this way it is possible to subdivide a reach into five parts without altering the remainder of the identification system. This flexibility requires a means of instructing the computer as to which integers represent a reach and which integers are unassigned. This is accomplished by a skip vector. This vector assigns a "1" for each integer representing a reach, and a "0" for the remaining integers. This is the basis for the "Does reach exist" branch.

9. Apply Multipliers

In this command, the multipliers described in item 7 are applied to the parameters for the reach being simulated.

All operations from this polnt on are within the first "DO" loop, and apply to a single reach. Succeeding reaches are handled in order, in an identical manner.

10. Compute Q From Upstream Reaches QIN

Flow entering the reach at the inlet is the flow leaving the immediate upstream reach. In the case of a junction, there would be two reaches immediately upstream. The fixed vector identifies the upstream reaches and calls for the upstream flows, which constitute QIN. In the case of an uppermost reach, there is no QIN. This is provided for by assigning the outflow from reach No. 1 as zero.

11. Compute Q Added Locally QADD

The hydrology of the Kansas river basin is such that a significant portion of the flow in the lower basin is contributed locally. For this reason it was necessary to include in the model a mechanism for adding local inflow.

The local inflow should reflect local hydrologic conditions. The best indication of these conditions is the flow in local tributaries. The QADD for a given reach is assumed to be a simple function of the flow in some tributary in the same vicinity. The details of the correlation studies which were performed to provide this functional relationship are described in Chapter II. The program requires the identification of the related stream gages and the coefficients for the QADD relationship. These data are included in the fixed vector. The flows at the various stream gages are given in the boundary condition vector.

The Q added locally is given by:

QADD = QAA(GAGE(JR1) + (GAGE(JR2)) + QAB,

in which

JR1, JR2 are the identifiers of the related gages.

In many cases there is only one related gage, in which case JR2 = 1, which is a zero gage.

QAA is the coefficient determined by correlation analysis.

QAB is an added constant which can be used to account for groundwater inflow in some reaches, or to account for local withdrawals or inputs from municipal or industrial operations.

The total flow in the reach, Q(I), is the sum of QIN and QADD.

12. Compute Average Velocity

The average velocity of flow in the reach is a function of the discharge. This functional relationship was determined from historical records of the U. S. Geological Survey, as described in Chapter III.

The coefficient VA and the exponent VB are included in the fixed vector for each reach.

13. Compute Average Hydraulic Depth H

The reoxygenation rate coefficient K2 is computed as a function of the hydraulic depth $H = \frac{A}{B}$, in which A is the cross-sectional area of the flow and B is the surface width. The hydraulic depth is a function of the discharge. This relationship was determined from the historical records of the U. S. Geological Survey and from cross-section data included in a flood-plain survey by the U. S. Corps of Engineers, as described in Chapter III.

The coefficient HA and the exponent HB are included in the fixed vector for each reach.

14. Compute K2 (Subroutine AK2)

The reoxygenation rate coefficient K2 is a function of the hydraulic conditions of the flow in the reach. Because of the numerous equations suggested by different authors, it was felt desirable to keep this computation as a subroutine which could be changed easily. Data supplied for this computation are average velocity V, hydraulic depth H, and average bed slope S.

15. **Compute Time of Flow Through Reach TF**

The time required for the flow to pass through the reach determines the condition of the organic pollution at the outlet and determines the lag time for dynamic routing.

$$TF = \frac{L}{16.4V},$$

in which 16.4 is a conversion factor to give TF in days from L in miles and V in ft/sec.

16. **Compute Initial BOD LA**

The oxygen-sag equation requires the initial organic load or BOD in mg/liter. The initial load at the beginning of a reach will include the remaining load entering from upstream reaches plus any new load added in the reach.

The load entering from upstream reaches is handled in the same way as QIN. To facilitate the mixing of loads from two upstream reaches at a junction, the loads are handled as lbs/day of BOD.

The new load added in the reach can come from several sources. Point sources such as sewer outfalls are given in the organic vector as ORGA values in lbs/day.

The wash load entering with surface runoff was assumed to be a function of the local inflow QADD. It was assumed that this function was of the form:

$$\text{Wash load} = a\ (QADD)^b,$$

and that a and b are constants. This relationship is a serious oversimplification, but represents a first approximation which is better than ignoring this potentially large source of pollution.

17. **Compute Initial DO Deficit DA**

The oxygen-sag equations require the initial dissolved-oxygen deficit in mg./l. This deficit can be evaluated by subtracting the amount of dissolved oxygen, DO, in the inflow from the saturation value. The mixing of the various sources of dissolved oxygen is facilitated by working with the lbs/day of oxygen contained in them. Sources of DO include the inflow from upstream reaches and the local inflow. The DO available in the inflow from an upstream reach is determined by subtracting the outflow deficit for that reach from the saturation value. The local inflow is assumed to be saturated with DO.

The saturation value SAT is a function of water temperature. For this model, the water temperature is assumed to be constant, giving a fixed value for SAT.

The initial DO deficit is determined by subtracting the total oxygen resource, converted to mg./l., from the saturated value SAT.

18. Compute Outlet BOD LTF (Subroutine LOADT, T = TF)

By means of the oxygen-sag equations it is possible to compute the magnitude of the remaining BOD at any time, T, in days, since the start of decay. The BOD remaining in the outflow from the reach is calculated by setting T = TF, where TF is the time of flow through the reach. This load, LTF, in mg./l. is converted to lbs./day, (L0(I)), for addition to the downstream reach. The computation is performed as a subroutine so that the form of the sag equation can be altered without altering the remainder of the program.

19. Compute Outlet DO Deficit DTF (Subroutine DEFT, T = TF)

The oxygen-sag equation gives the DO deficit in mg./l. at any time, T, in days after the beginning of decay. Conditions at the outlet of the reach can be determined by setting T = TF. The deficit DTF thus calculated is then subtracted from the saturation value

SAT to give the dissolved oxygen in the outflow. This difference in mg./l. is then converted to DO(I) lbs./day for addition to the downstream reach.

20. Compute Time to Critical Sag TC (Subroutine TCR)

The oxygen-sag equation relates oxygen deficit to elapsed time. This deficit reaches a maximum value at some critical time, TC, after which the deficit decreases. The critical time, TC, is evaluated by setting the derivative of the sag equation with respect to time equal to zero. With low initial loads and high reoxygenation rates, it is possible to end up with a sag relationship which is continuously decreasing so that TC becomes negative. For this condition, the equation breaks down, and TC is set equal to TF.

21. Compare TC with TF

The primary object of the program is to determine the maximum dissolved oxygen deficit occurring in the reach. If TC is greater than TF, then the deficit occurring at the outlet is the maximum deficit. In this case, the maximum deficit DMAX is set equal to DTF.

If TC is less than TF, the maximum deficit does not occur at the outlet. Rather, the maximum deficit occurs at the critical point where T = TC. In this case it is necessary to compute the critical deficit DTC by calling subroutine DEFT with T = TC. DMAX is then equated with DTC.

22. Compare DMAX with LIMIT

Regardless of which method was used to compute DMAX, it is now necessary to compare DMAX with some pre-assigned allowable limit value. If the maximum deficit, DMAX, occurring in the reach is less than the allowable limit, then the reach does not have a DO problem, and the program moves on to the conservative pollution calculations.

If DMAX is greater than the limit, then this condition is noted in the output by writing out the identity of the reach, I; the magnitude of the deficit, DMAX; and the location within the reach of the critical point.

23. Compute Conservative From Upstream Reaches CIN

Conservative pollutants are routed through the river system in the same manner as the water.

24. Compute Conservative Added Locally

The mechanism by which a conservative pollutant enters the river system is both complex and variable. For this model, it was assumed that the amount of conservative added in a reach (CADD) in tons/day is some function of the local inflow, QADD. As discussed in item 4, the coefficients and exponents defining this functional relationship for each reach are given in the conservative vector. Given QADD and the constants for the equation, the conservative added locally is computed.

25. Compute Conservative Concentration CONC

The parameter of concern with regard to a conservative pollutant is the concentration in mg./l. rather than the load in tons/day. This parameter is determined by dividing the load in tons/day by the flow in cfs multiplied by the conversion factor .0027.

26. Compare Concentration with Limit

As with organic pollution, if the concentration of a conservative pollutant CONC is less than the limit, then the conservative pollutant is not a problem in that reach, and the program moves on to the next reach.

If the concentration CONC is greater than the limit, then this condition is noted in the output by writing out the identity of the reach, I, and the magnitude of the concentration CONC.

27. **Iterate to Next Reach**

Having completed the analysis of that reach, the program moves on to the next reach and repeats the above operations. This procedure is repeated reach by reach until all reaches have been analyzed.

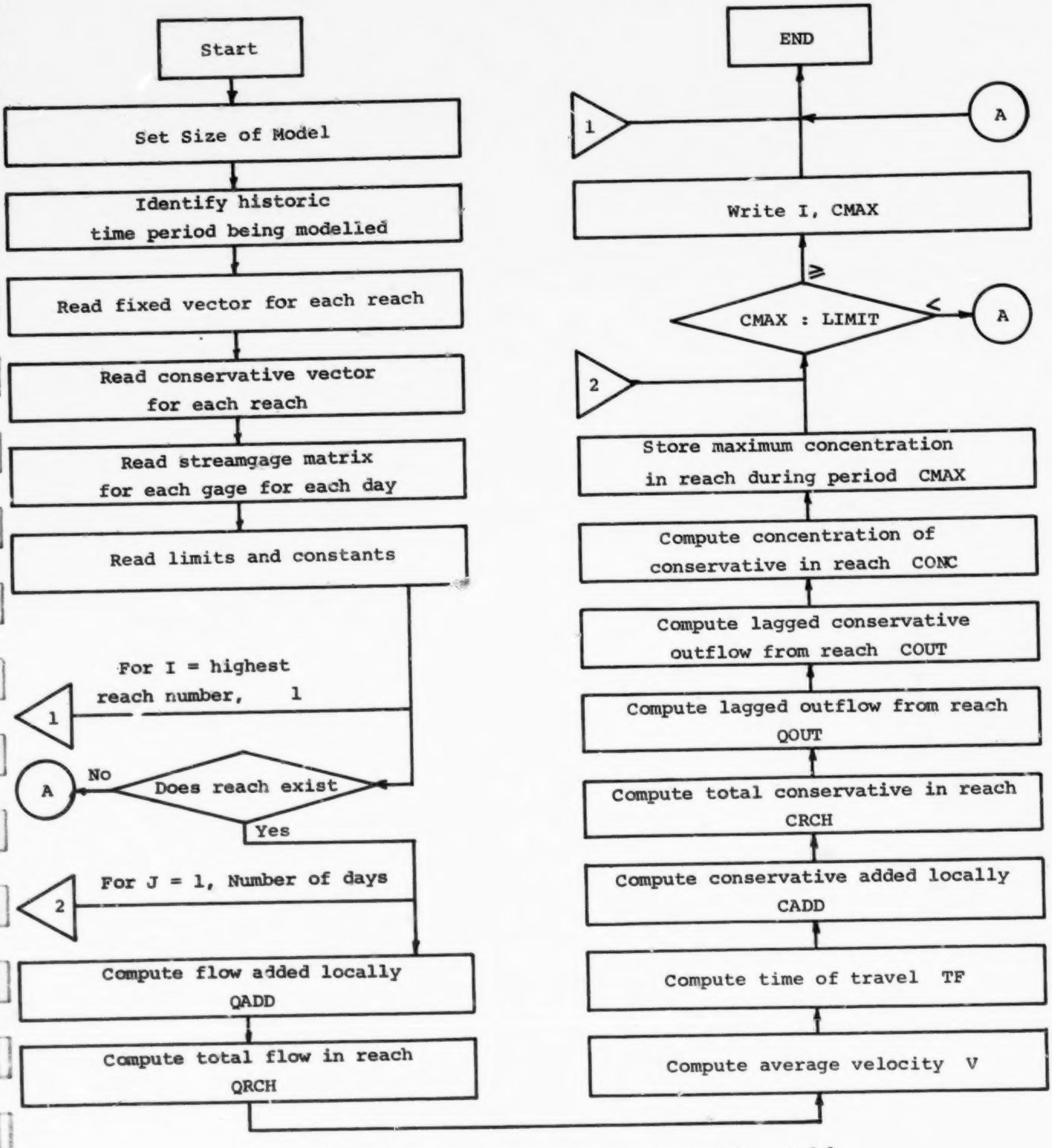

Start
Set Size of Model
Identify historic time period being modelled
Read fixed vector for each reach
Read conservative vector for each reach
Read streamgage matrix for each gage for each day
Read limits and constants
For I = highest reach number, 1
1
No
A
Does reach exist
Yes
For J = 1, Number of days
2
Compute flow added locally QADD
Compute total flow in reach QRCH
Compute average velocity V
Compute time of travel TF
Compute conservative added locally CADD
Compute total conservative in reach CRCH
Compute lagged outflow from reach QOUT
Compute lagged conservative outflow from reach COUT
Compute concentration of conservative in reach CONC
Store maximum concentration in reach during period CMAX
2
CMAX : LIMIT
<
A
≥
Write I, CMAX
1
A
END

Figure 20 Flow Diagram for the Dynamic Conservative Model

Dynamic Model for Conservative Pollutants

The Dynamic Model for Conservative Pollutants contains many operations which are identical to those described in the Quasi-Steady-State Model. The operations which differ between the models or are specific to the dynamic model are discussed below. The number preceding the description indicates the location of the operation in the flow diagram shown in Figure 20.

1. Set Size of Model

The size of the river system to be modelled is established by the number of reach designations, and the number of stream gages in the system. These numbers are read in at the beginning to reserve the required amount of storage in the computer.

The size of the dynamic model includes the additional parameter "Number of Days" which sets the length of the historic record under simulation. The size of this term has a pronounced effect on the required storage capacity because it is a multiplier of all of the double-subscripted variables. The actual limit on the number of days is the storage capacity of the computer.

2. Identify Historic Time Period Being Modelled

Each historic period presents a unique set of boundary conditions. These periods are identified by calendar date for future reference.

5. Read Stream-gage Matrix for Each Gage for Each Day

The dynamic model acknowledges the variable nature of stream flow data by reading in a hydrograph for each stream gage. A double-subscripting scheme identifies the gage and the day for each flow. In this way it is possible to store a hydrograph for each gage rather than a single value. This discrete series of daily mean discharges is the foundation of the dynamic model.

8. Iteration Through All Days of Period

The days of the period are numbered consecutively from one to the number of days ND. The program starts at day one and proceeds day by day to the end of the period.

10. Compute Total Flow in Reach

The flow in the reach will be the sum of the flows entering from upstream reaches and the flow added locally, QADD. As in the Quasi-Steady-State model, the identities of the contributing upstream reaches are contained in the fixed vector. In the dynamic model the program must call for the correct day as well as for the correct reach. For this reason, QOUT is double subscripted.

14. Compute Total Conservative in Reach CRCH

The total amount of conservative pollutant passing through the reach in tons per day is the sum of the load added locally CADD and the load entering from upstream reaches. The load for upstream reaches is determined in the same manner as the water entering from those reaches. In the dynamic model this requires the program to call for the conservative outflow COUT from the appropriate reaches for the day under consideration. For this reason, COUT must be double subscripted.

15. Compute Lagged Outflow from Reach QOUT

The lagged outflow from the reach under consideration will become the inflow for some downstream reach. In the dynamic model this outflow is a hydrograph rather than a single value. This hydrograph is first determined at the beginning of the reach and is then lagged to the outlet of the reach. The present form of the model does not allow for any modification of the hydrograph due to channel storage. The lag time is assumed to be equal to the time of flow. The program achieves this simple routing by determining inlet conditions for a given day and distributing the resulting discrete daily flow between several subsequent days at the outlet. This distribution is based on the time of flow associated with inlet conditions for the given day. The program progresses day by day throughout the historic period until the resulting outflow hydrograph has been produced. The hydrograph is a series of discrete mean daily flows which is thus available as input to downstream reach calculations. When lag times of several days are encountered, this routine will give meaningless data for a short period at the beginning and end of the hydrograph.

16. Compute Lagged Conservative Outflow from Reach COUT

The dynamic model produces a "hydrograph" of conservative outflow values in the same way as water outflow.

17. Compute Concentration of Conservative in Reach CONC

The model will compute the concentration of the conservative pollutant for each day of the period under study. The concentration in mg./l. is determined from the load in tons/day and the flow in the reach in cfs by the relationship

$$\text{CONC} = \text{COUT} \div \text{QOUT} \times .0027$$

18. Select Maximum Concentration in Reach During Period CMAX

For control purposes, only the maximum concentration occurring in the reach during the period under study is pertinent. The program selects this maximum CMAX.

19. Iterate to Next Day

Having completed the analysis for that day, the program moves on to the next day and repeats the above operations. This procedure is repeated day by day until all outflow "hydrographs" have been computed.

20. **CMAX: LIMIT**

The conservative pollutant is considered a problem only if the concentration of the pollutant exceeds some externally extablished limit concentration. If the maximum concentration in the reach CMAX is equal to or greater than the allowable limit, the magnitude of CMAX and the reach identity are printed in the output. Otherwise there is no output for the reach.

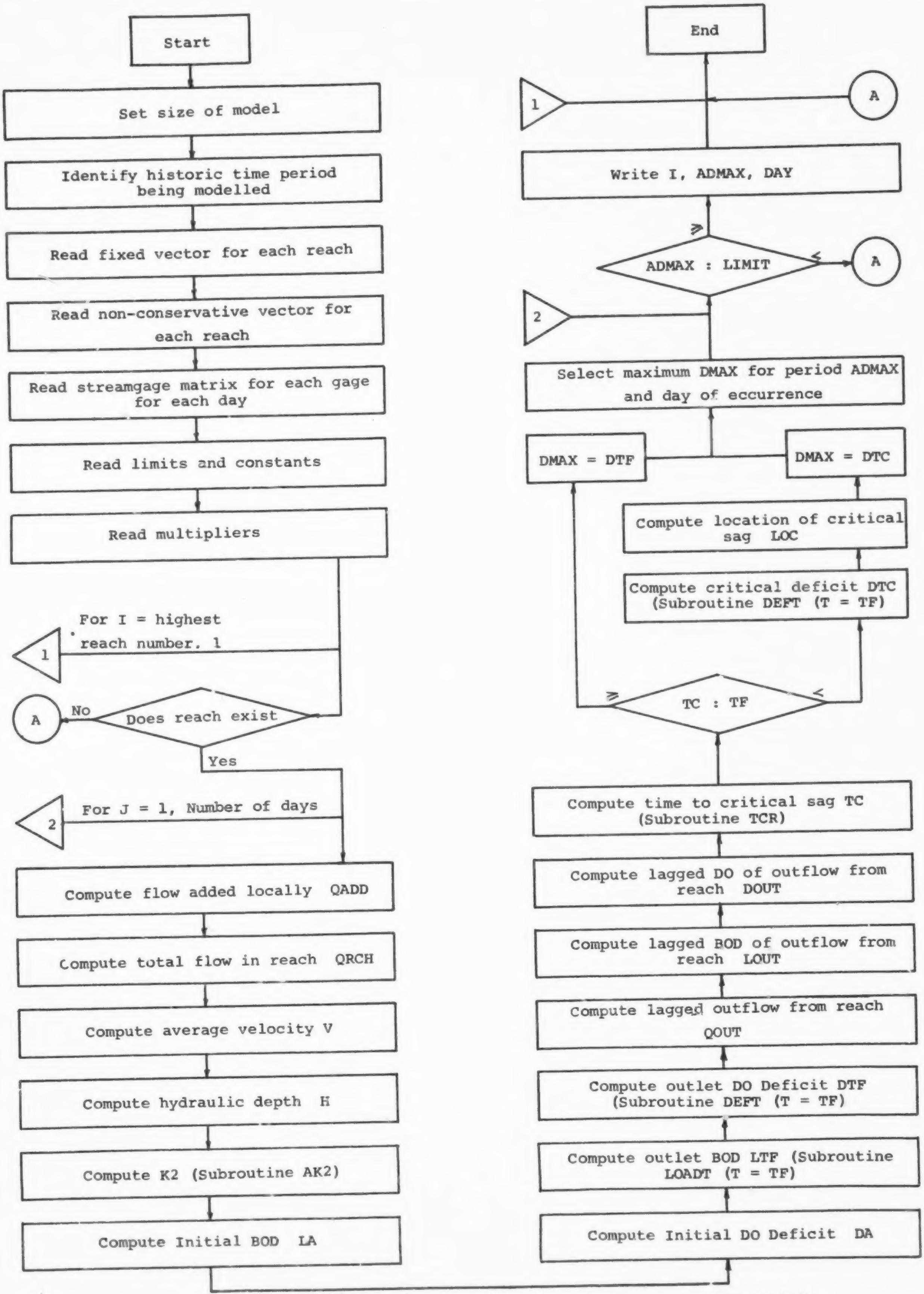

Start
Set size of model
Identify historic time period being modelled
Read fixed vector for each reach
Read non-conservative vector for each reach
Read streamgage matrix for each gage for each day
Read limits and constants
Read multipliers
For I = highest reach number. 1
1
A
No
Does reach exist
Yes
For J = 1, Number of days
2
Compute flow added locally QADD
Compute total flow in reach QRCH
Compute average velocity V
Compute hydraulic depth H
Compute K2 (Subroutine AK2)
Compute Initial BOD LA
Compute Initial DO Deficit DA
Compute outlet BOD LTF (Subroutine LOADT (T = TF)
Compute outlet DO Deficit DTF (Subroutine DEFT (T = TF)
Compute lagged outflow from reach QOUT
Compute lagged BOD of outflow from reach LOUT
Compute lagged DO of outflow from reach DOUT
Compute time to critical sag TC (Subroutine TCR)
TC : TF
≥
<
Compute critical deficit DTC (Subroutine DEFT (T = TF)
Compute location of critical sag LOC
DMAX = DTF
DMAX = DTC
Select maximum DMAX for period ADMAX and day of ecurrence
2
ADMAX : LIMIT
≥
<
A
Write I, ADMAX, DAY
1
A
End

Figure 21 Flow Diagram for the Dynamic Non-Conservative Model

Dynamic Model for Non-Conservative Pollutants

The dynamic model for non-conservative pollutants contains many operations which are identical to those described in the Quasi-Steady-State Model. The operations which differ between the models or are specific to the dynamic model are discussed below. The number preceding the description indicates the location of the operation in the flow diagram shown in Figure 21.

1. Set Size of Model

The size of the river system to be modelled is established by the number of reach designations, and the number of stream-gages in the system. These numbers are read in at the beginning to reserve the required amount of storage in the computer.

2. Identify Historic Time Period Being Modelled

Each historic period presents a unique set of boundary conditions. These periods are identified by calendar date for future reference.

5. Read Stream-gage Matrix for Each Gage for Each Day

The dynamic model acknowledges the variable nature of stream flow data by reading in hydrographs for each stream gage. A double subscripting scheme identifies the gage and the day for each flow. In this way it is possible to store a hydrograph for each gage rather than a single value.

This discrete series of daily mean discharges is the foundation of the dynamic model.

9. Iteration Through All Days of Period

The days of the period are numbered consecutively from one to the number of days ND. The program starts at day one and proceeds day by day to the end of the period.

11. Compute Total Flow in Reach

The flow in the reach will be the sum of the flows entering from upstream reaches and the flow added locally, QADD. As in the Quasi-Steady-State model, the identities of the contributing upstream reaches are contained in the fixed vector. In the dynamic model the program must call for the correct day as well as for the correct reach. For this reason, QOUT is double subscripted.

19. Compute Lagged Outflow from Reach QOUT

The lagged outflow from the reach under consideration will become the inflow for some downstream reach. In the dynamic model this outflow is a hydrograph rather than a single value. This hydrograph is first determined at the beginning of the reach and is then lagged to the outlet of the reach. The present form of the model does not allow for any modification of the hydrograph due to channel storage. The lag time is assumed to be equal to the time of flow. The program achieves this simple routing by determining inlet conditions for a given day and distributing the resulting discrete daily flow between several subsequent days at the outlet. This distribution is based on the time of flow associated with inlet conditions for the given day. The program progresses day by day throughout the historic period until the resulting outflow hydrograph has been produced. The hydrograph is a series of discrete mean daily flows which thus is available as input to downstream reach calculations. When lag times of several days are encountered, this routine will give meaningless data for a short period at the beginning and end of the hydrograph.

20. Compute Lagged BOD of Outflow from Reach LOUT

The outlet BOD computed by the oxygen-sag equations is based on the hydraulic conditions occurring at the reach inlet on the day under consideration. The outlet BOD for a given reach will become the inlet BOD for the reach downstream. It is necessary to develop a "hydrograph" of BOD values for each reach. This is achieved by determining outlet conditions for a given day and distributing the resulting discrete mean daily BOD values which are thus available as input to downstream reach calculations.

21. Compute Lagged DO of Outflow from Reach DOUT

The outlet DO deficit computed by the oxygen-sag equation is used to determine the remaining dissolved oxygen DO. This oxygen resource is then lagged in a manner similar to the BOD. The resulting "hydrograph" of DO is thus available as input to downstream reach calculations of initial DO deficit.

24. Select Maximum DMAX for Period ADMAX

The oxygen deficit of prime interest is the maximum value occurring in the reach during the time period under study. The program selects the maximum daily DMAX, ADMAX.

25. Iterate to Next Day

Having completed the analysis for that day, the program moves on to the next day and repeats the above operations. This procedure is repeated day by day until all outflow "hydrographs" have been computed.

26.. ADMAX: LIMIT

If the maximum deficit ADMAX occurring in the reach during the time period is greater than or equal to the established limit value, then this condition is noted in the output by writing the reach identity, the value of ADMAX, and the day on which the maximum deficit occurred.

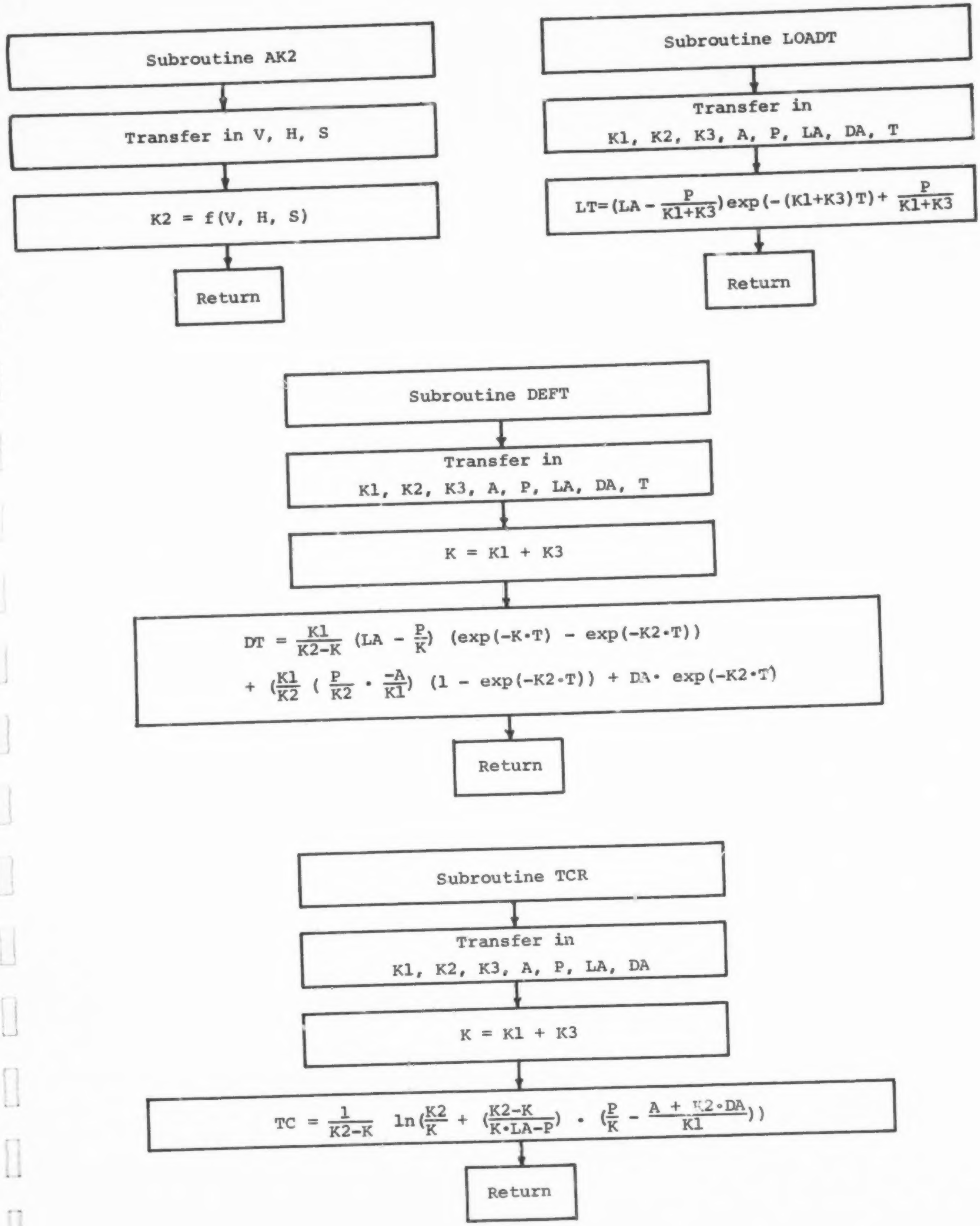

Subroutine AK2
Transfer in V, H, S
K2 = f(V, H, S)
Return
Subroutine LOADT
Transfer in
K1, K2, K3, A, P, LA, DA, T
$LT=(LA-\frac{P}{K1+K3})\exp(-(K1+K3)T)+\frac{P}{K1+K3}$
Return
Subroutine DEFT
Transfer in
K1, K2, K3, A, P, LA, DA, T
K = K1 + K3
$DT=\frac{K1}{K2-K}(LA-\frac{P}{K})(\exp(-K\cdot T)-\exp(-K2\cdot T))$
$+(\frac{K1}{K2}(\frac{P}{K2}\cdot\frac{-A}{K1})(1-\exp(-K2\cdot T))+DA\cdot\exp(-K2\cdot T)$
Return
Subroutine TCR
Transfer in
K1, K2, K3, A, P, LA, DA
K = K1 + K3
$TC=\frac{1}{K2-K}\ \ln(\frac{K2}{K}+(\frac{K2-K}{K\cdot LA-P})\cdot(\frac{P}{K}-\frac{A+K2\cdot DA}{K1}))$
Return

Figure 22 Computor Program Subroutines

Subroutines

The computer programs for the three models have referred to several subroutines. The FORTRAN language of the computer provides for this method of calling for a particular computation by a set of commands which are not a part of the main program. There are several advantages to the use of subroutines. One is the ability to use the same subroutine to compute several different outputs. Another is the ease with which a subroutine may be changed without disruption of the main program. Both of these advantages have been exploited in this project. The DEFT subroutine is used first to compute DTF and then used a second time to compute DTC. The AK2 subroutine is a good example of a function which likely will be changed several times.

The four subroutines used in these models are shown in Figure 22. The necessary data are first transferred into the subroutine from the main program. After the computation is complete the answer is transferred back to the main program and control is returned to the main program. It should be noted that the symbolism need not be the same in the subroutine as in the main program. For instance, DT becomes DTF when T is TF.

VII. Model Verification

The hydrologic portion of the Lower Kansas River Basin Model was tested by using historic streamflow data at the tributary gaging stations as input to the model and determining how well the model simulated the actual streamflow occurring on the main stream stations. The dynamic model was used in these studies because there were very few periods in the historic record where the streamflows were constant over a long enough period of time to satisfy the quasi-steady-state model.

The six time periods tabulated in Table 7 were used for model verification. In selecting these periods, an effort was made to choose records in which flow rises occurred on the tributary streams. This was done to permit evaluation of the model in routing a sudden increase in discharge through the river system. The results obtained at four main stream stations during two of the flow periods are shown in Figures 23 through 30. In these graphs, the solid line is the actual streamflow observed at the gaging station, and the broken line is the simulated streamflow obtained from the model. It should be noted that the same vertical scale is not used for all of the Figures. The apparent differences between observed and predicted discharge are therefore relative to the vertical scale. The results illustrated in Figures 23 through 30 are typical of the results obtained for all six verification periods.

Table 7

Discharge Records Used in Verifying the Lower Kansas River Basin Model

February 15 - March 31, 1961	November 1 - December 12, 1962
April 1 - May 4, 1961	June 1 - July 15, 1963
March 1 - April 15, 1962	July 20 - August 31, 1965

The observed and predicted discharges for the Solomon River at Niles compare very well during the 1962 flow period. This is not true for the 1965 flow period. The reason for the discrepancies both in magnitude and timing during the 1965 flow period is that the station is located in the western portion of the model area, and the QADD functions are based on only a few tributary gages. Under these conditions, it is possible to completely miss a thunderstorm which would contribute a significant amount of runoff to the stream. The errors in timing of the peak discharge are dependent upon which direction the storm is moving. There is no way of compensating for individual storm movement in the model at the present time.

It is worthwhile to note, however, that the discrepancies in both timing and magnitude which occasionally occur in the upstream reaches of the model appear to adjust themselves as the upstream discharge passes down through the river system.

The discharge hydrograph for the Kansas River at Bonner Springs depicted in Figure 30 illustrates another problem in the model which is associated with the groundwater effluent found in the lower reaches of the river system. Considerable water is released from bank storage when the Kansas River recedes from high stage to low stage in a short period of time. The constant QADD function used in the model at the present time to account for groundwater effluent to the river system does not compensate for the inflow from bank storage. This contribution could be accounted for if groundwater observation wells were available in the alluvial aquifer adjacent to the river, by developing a correlation between the water level in the wells and the discharge in the river. This modification does not appear warranted, however.

The conservative water quality vectors in the Lower Kansas River Basin Model were evaluated for the concentration of chloride occurring on the main stream stations. All six time periods listed in Table 7 were used in this study. The results obtained with the chloride vectors listed in Appendix F are given in Figures 31 through 34. The observed data points given in these Figures are grab sample values obtained from the appropriate Water Supply Papers of the U. S. Geological Survey. All available observed data points are plotted on the graphs. Only one point has been plotted in cases of identical predicted values. The chloride vectors were optimized to obtain the results shown in the Figures. This was accomplished by initially using the vectors obtained by the correlation method outlined in Chapter IV and adjusting the vertical intercept of the vectors, such as the ones illustrated in Figure 11, until a satisfactory grouping of the predicted values was obtained. It was rarely necessary to adjust the slope of the vector. This correction was necessary because the method outlined in Chapter III does not consider the time of flow in the river system. A more satisfactory method of obtaining the conservative vectors would be to use the mathematical model to evaluate them within the computer from the observed data. A program for performing this computation is under development at the present time.

Time did not permit the testing of the chloride vectors given in Appendix F on discharge records other than those given in Table 7. However, it is felt that the vectors given in the appendix are reliable because of the wide variety of discharge patterns available in the streamflow records used.

It is significant to note that the same general trend of improved correlation between the predicted value and the observed value at the downstream stations observed with the discharge verification studies also appears to hold true with the conservative water quality.

Verification of the non-conservative water quality aspects of the model was not possible because of the lack of observed data. It is anticipated that the field observation program conducted in 1966 will provide much of the information needed to establish the values of many of the parameters entering the oxygen-sag equation developed by Camp(11) and Dobbins(12).

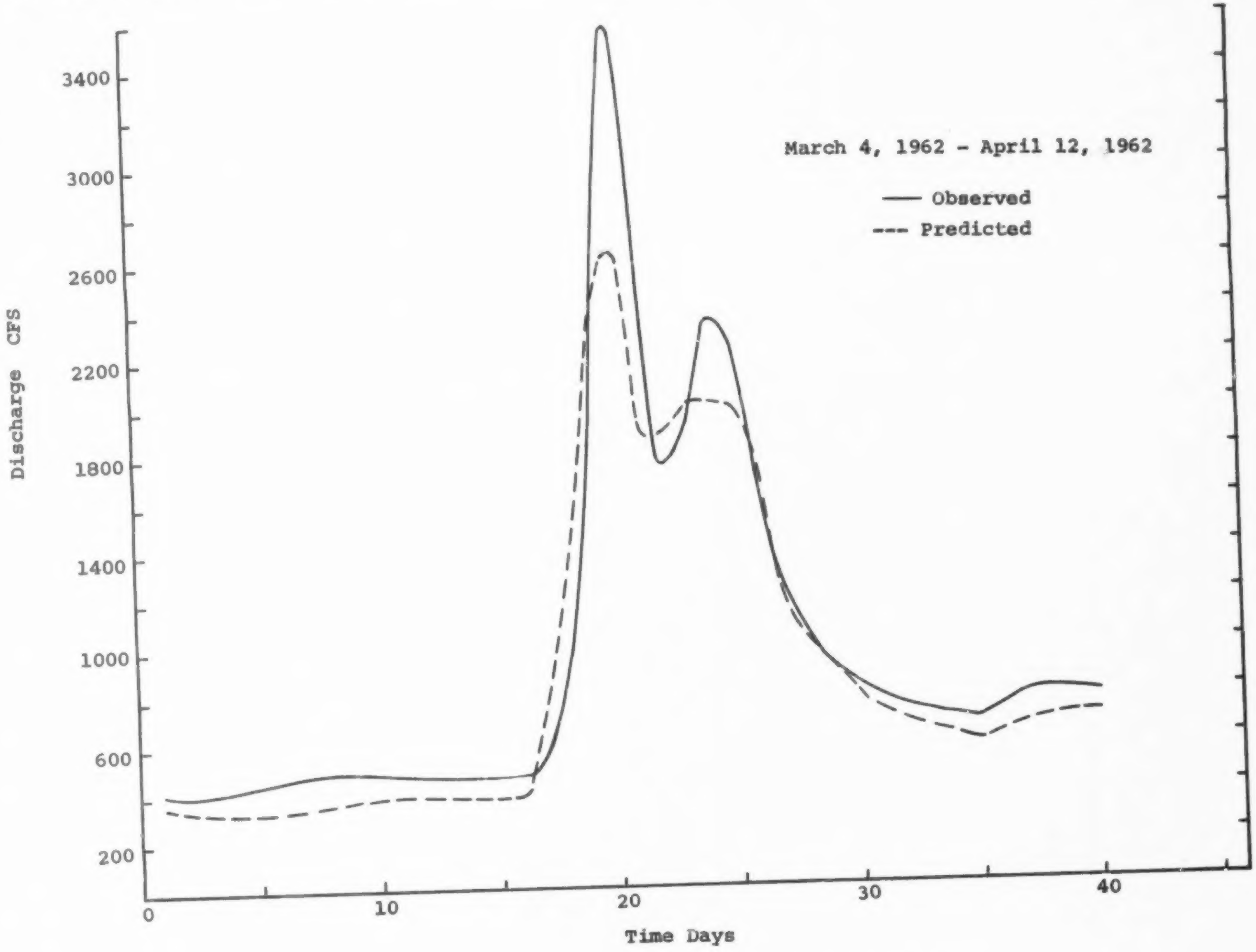


Figure 23 Observed and Predicted Discharge
Gaging Station Number 21
Solomon River at Niles

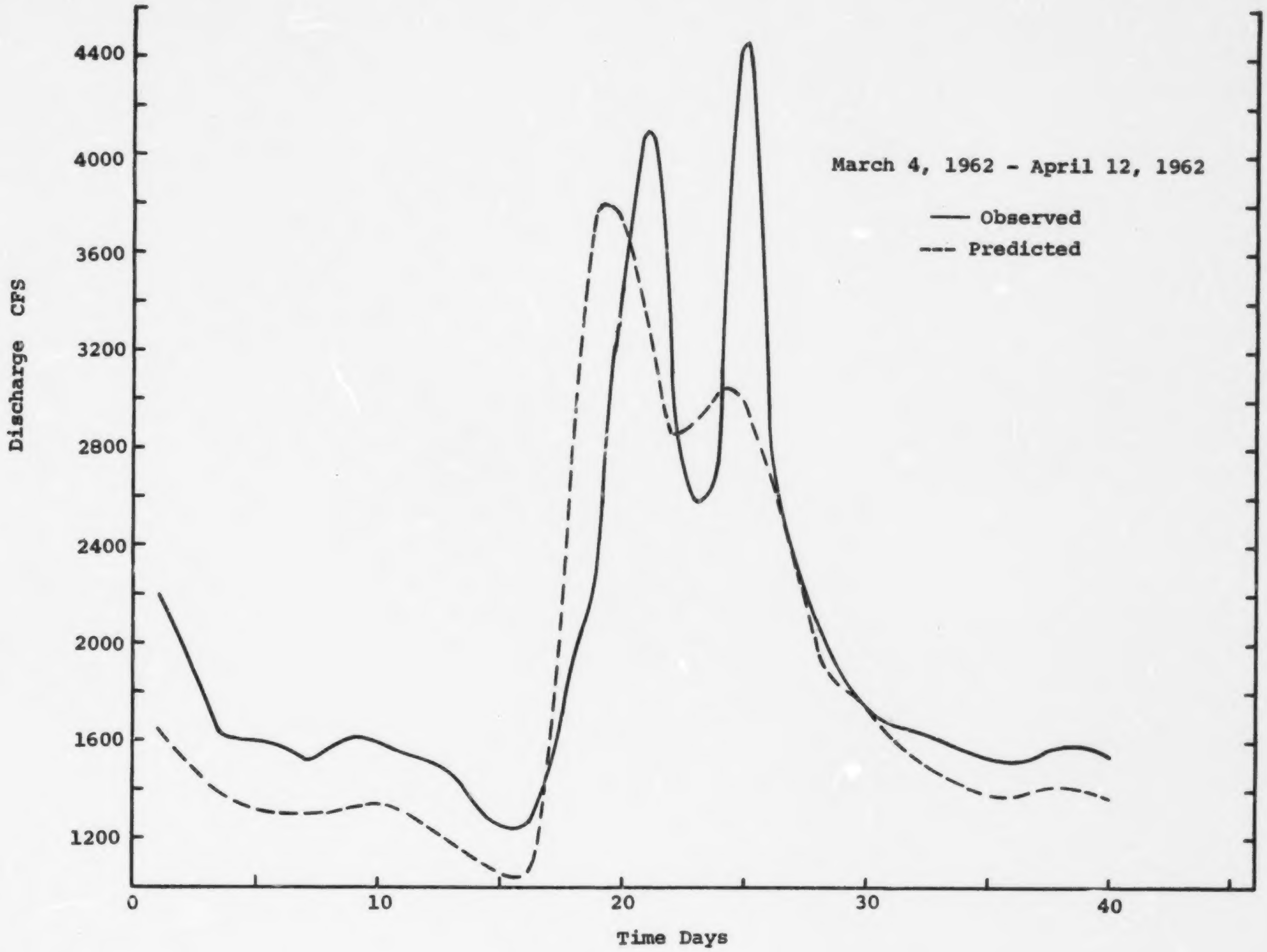


Figure 24 Observed and Predicted Discharge
Gaging Station Number 19
Smoky Hill River at Enterprise

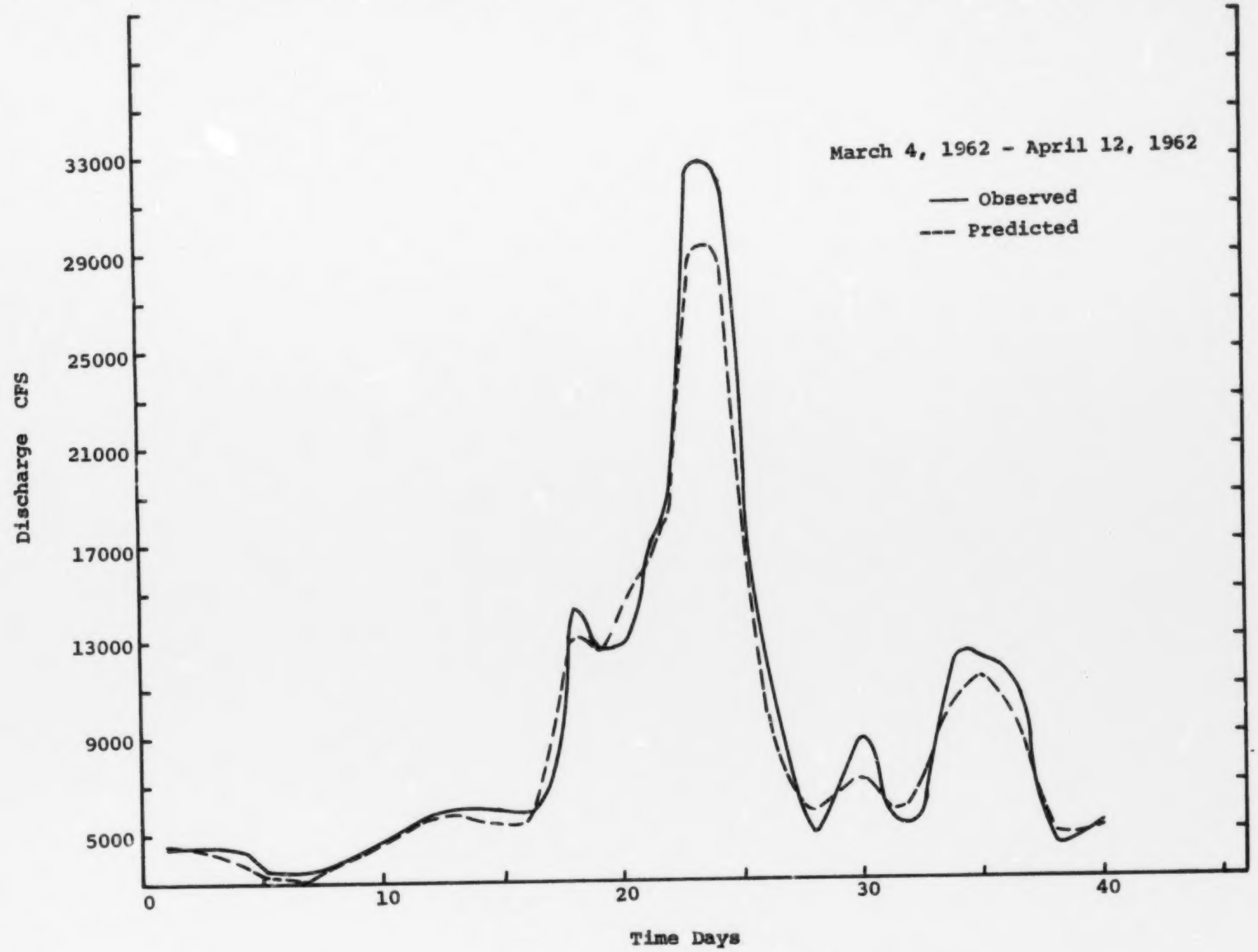


Figure 25 Observed and Predicted Discharge
Gaging Station Number 11
Kansas River at Wamego

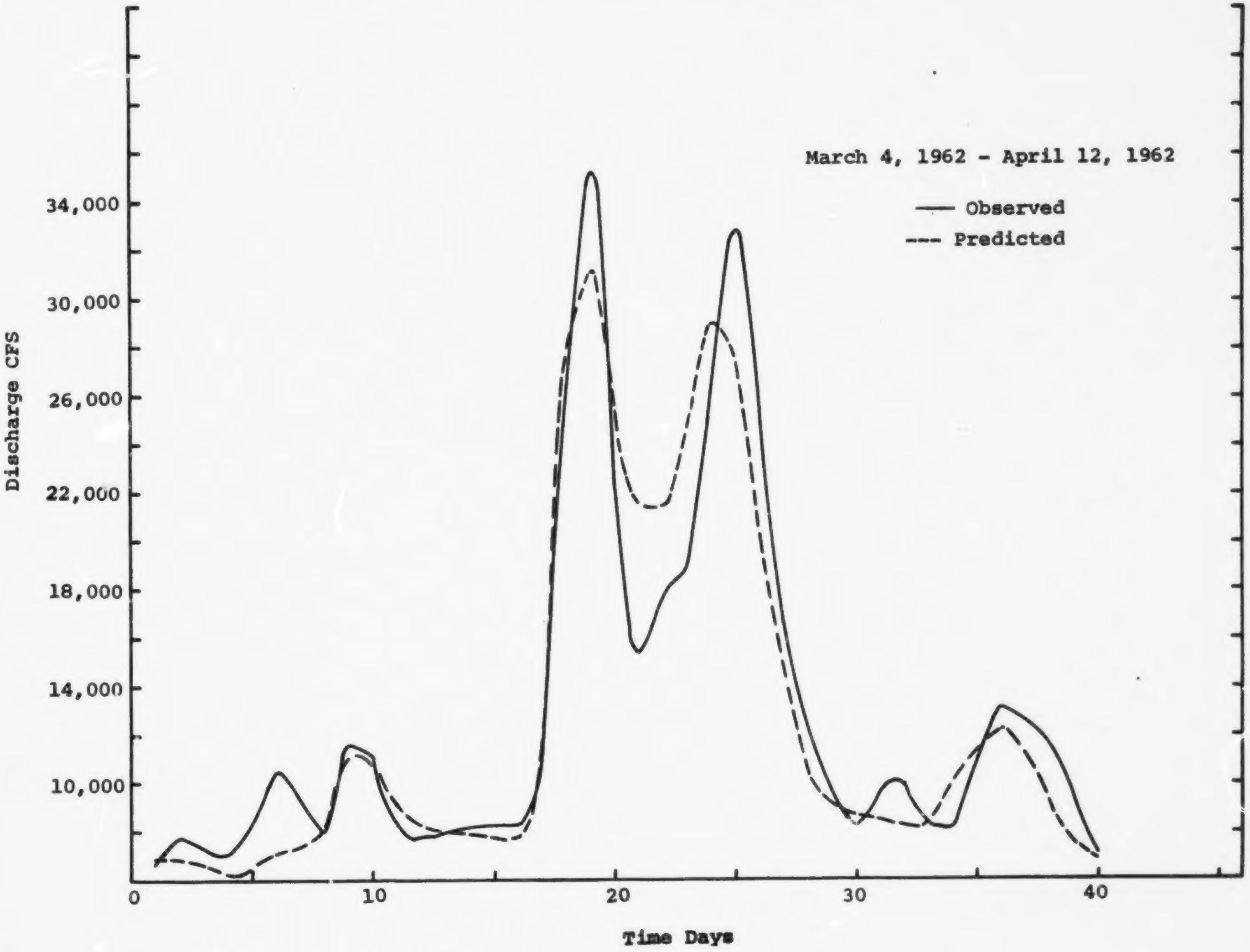


Figure 26 Observed and Predicted Discharge
Gaging Station Number 1
Kansas River at Bonner Springs

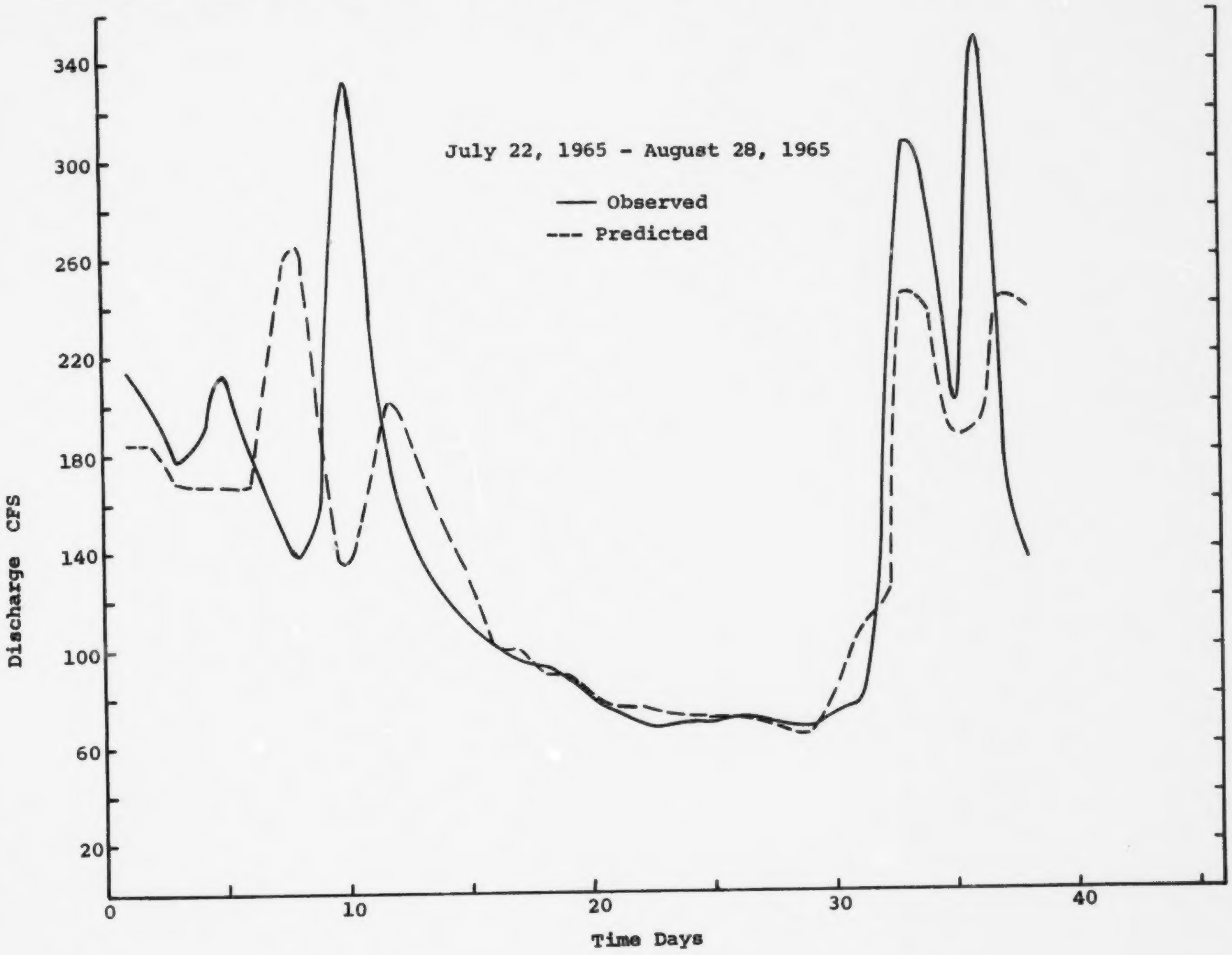


Figure 27 Observed and Predicted Discharge
Gaging Station Number 21
Solomon River at Niles

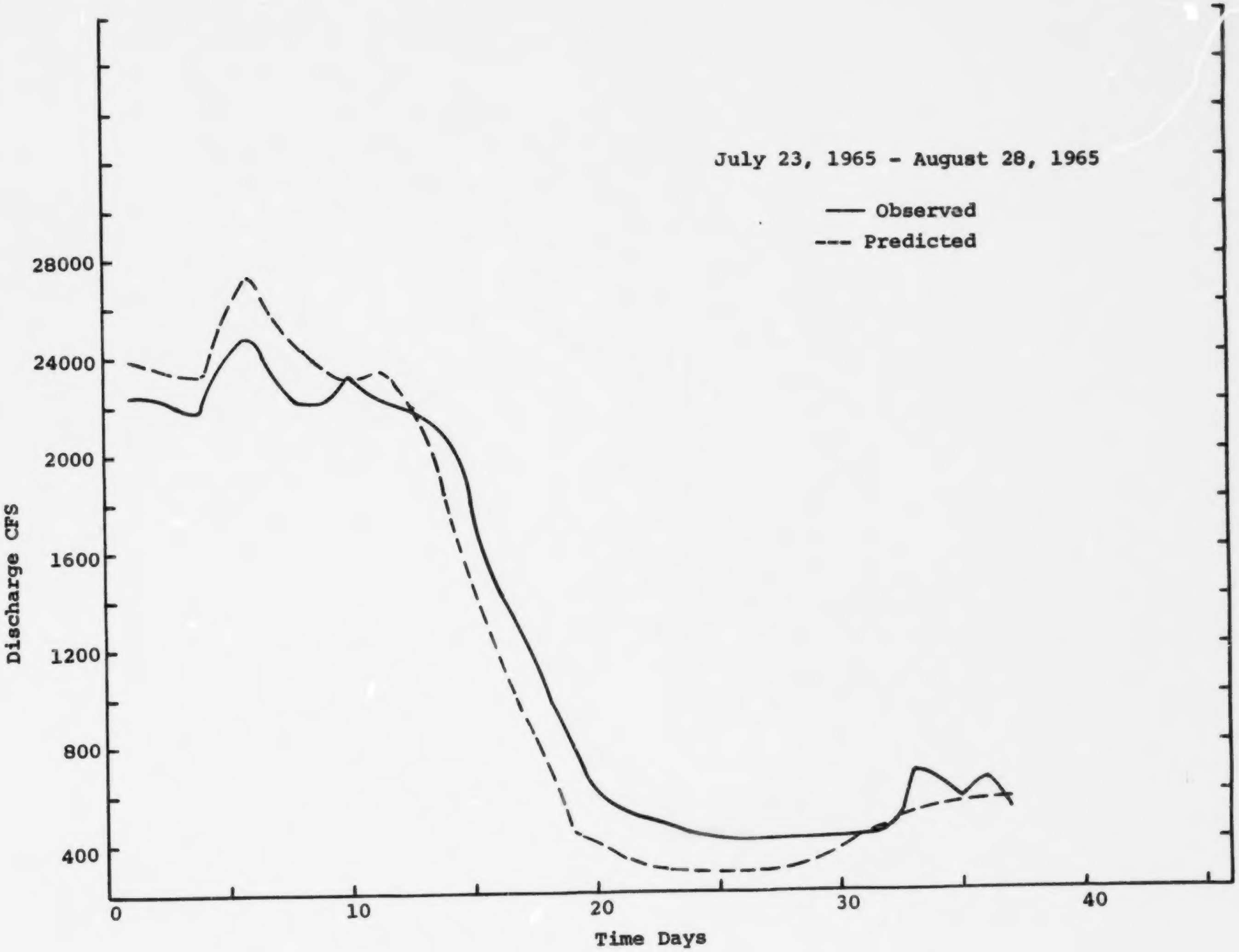


Figure 28 Observed and Predicted Discharge
Gaging Station Number 19
Smoky Hill River at Enterprise

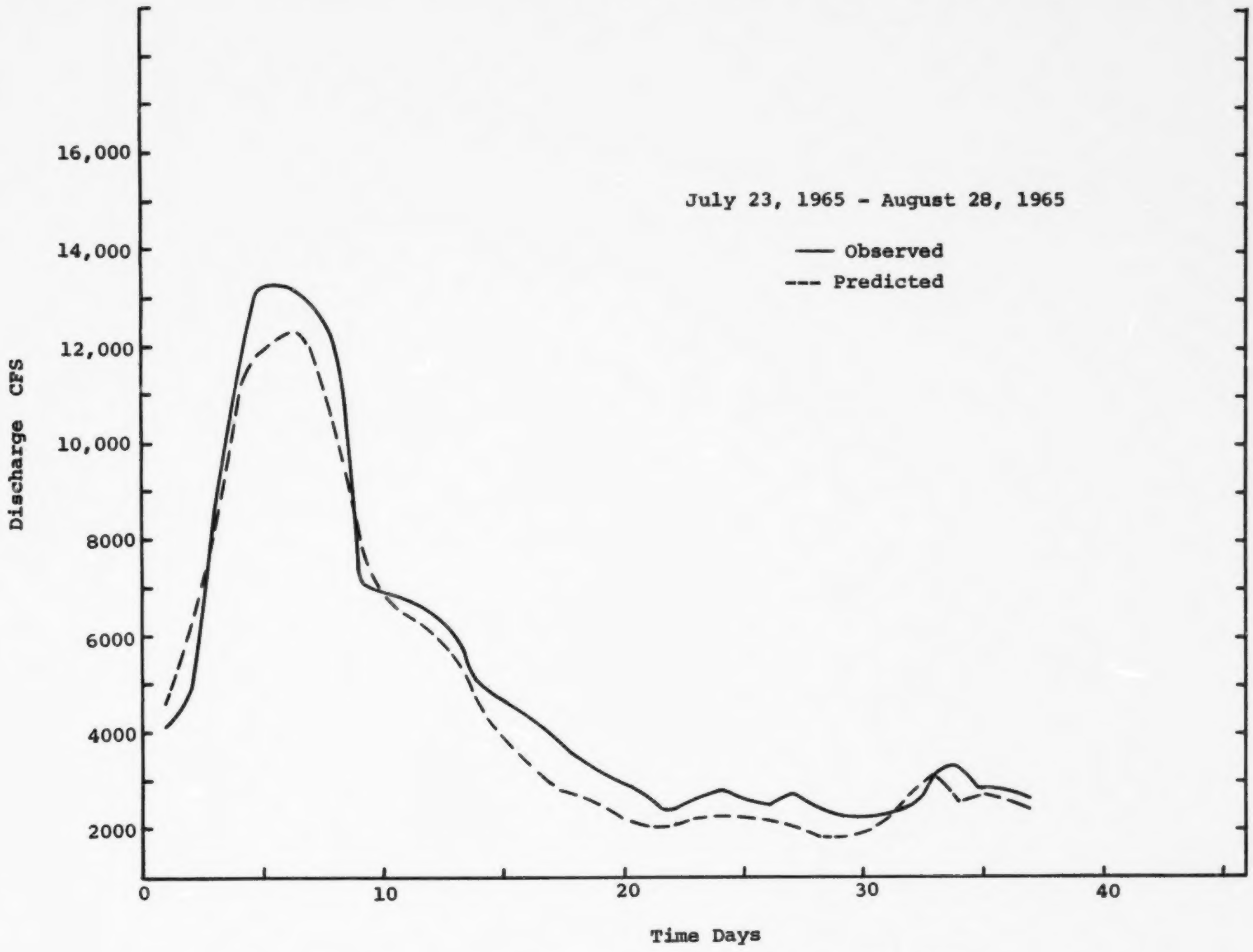


Figure 29 Observed and Predicted Discharge
Gaging Station Number 11
Kansas River at Wamego

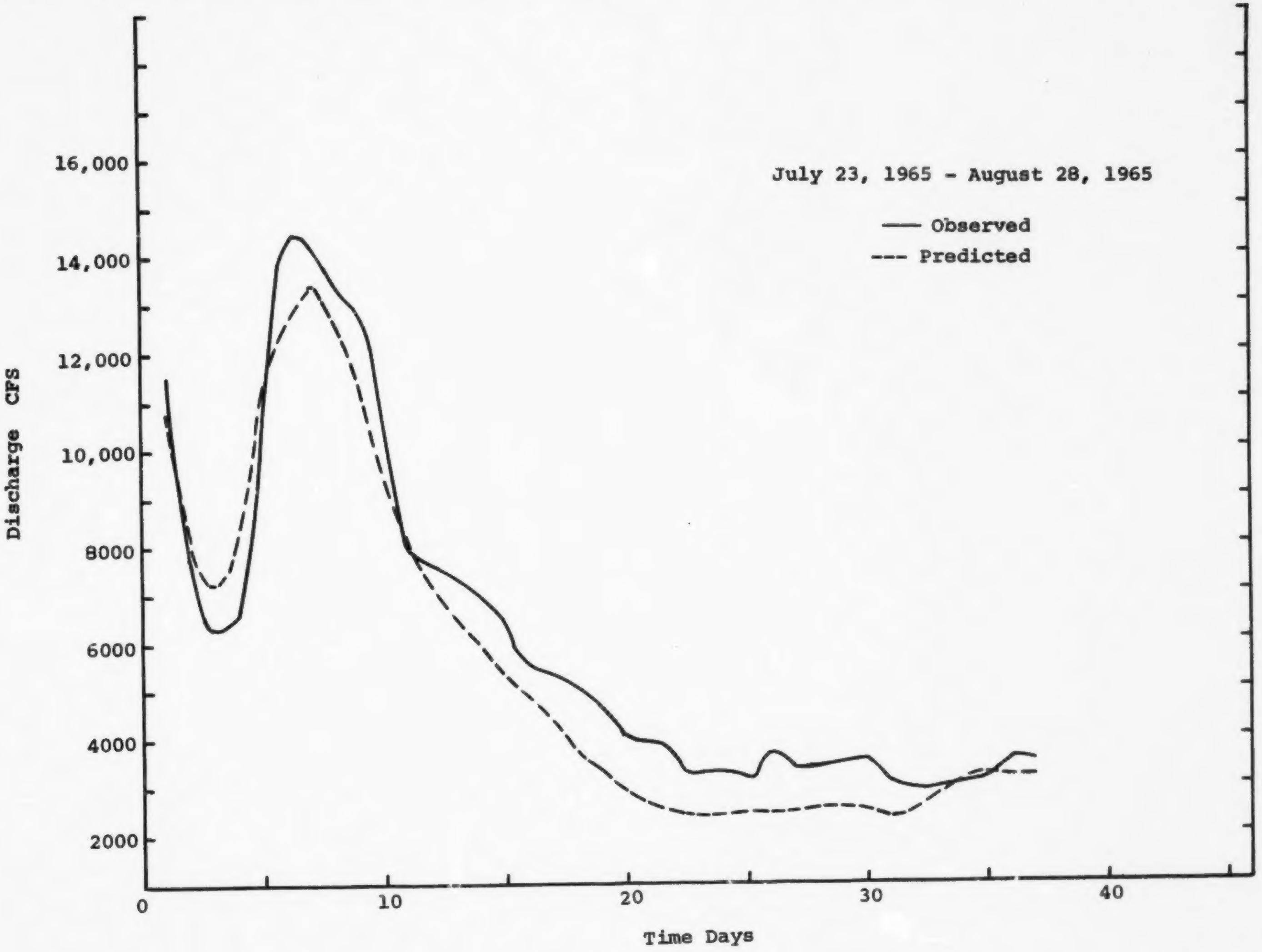


Figure 30 Observed and Predicted Discharge
Gaging Station Number 1
Kansas River at Bonner Springs

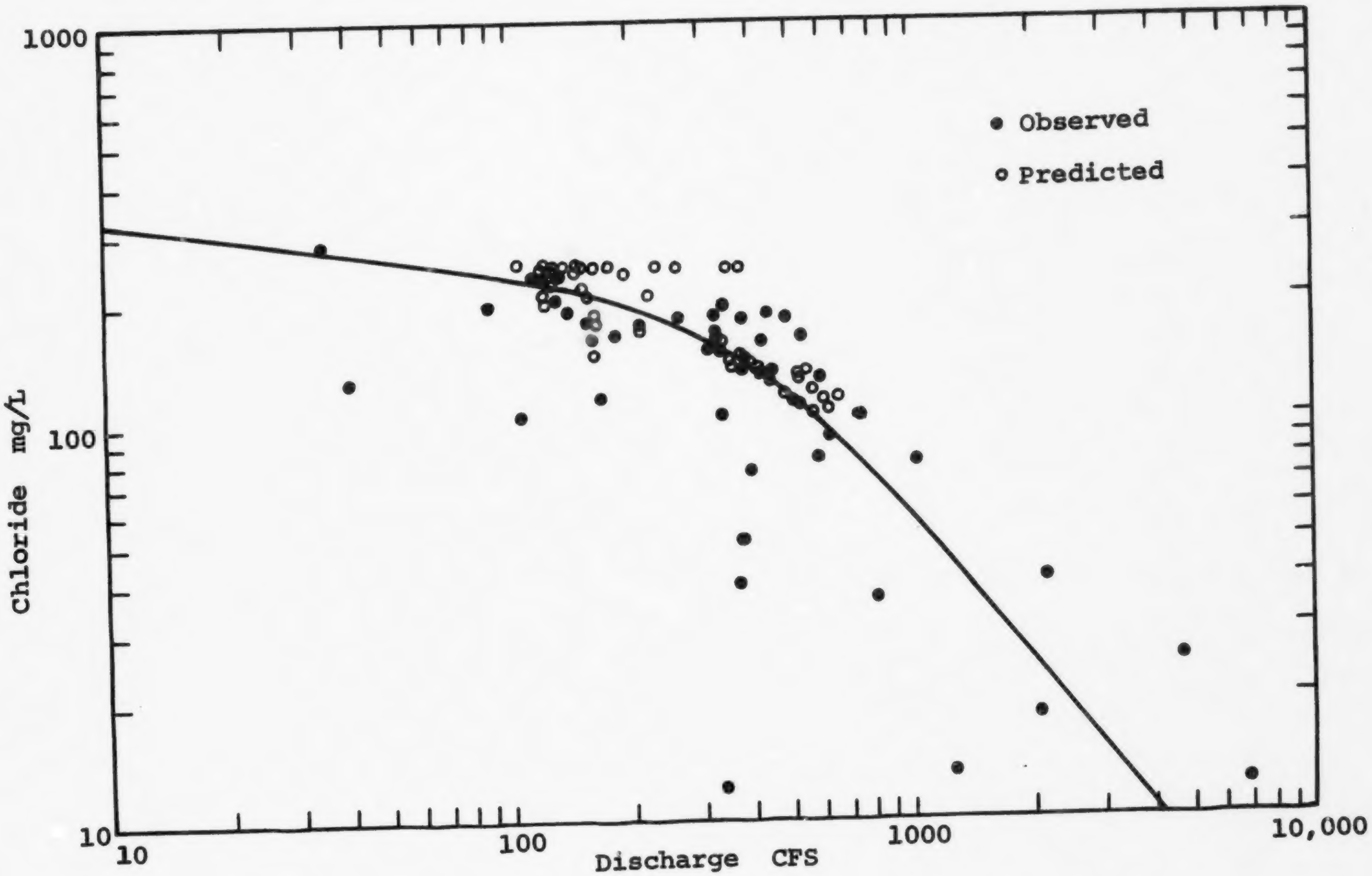


Figure 31 Observed and Predicted Chloride Concentration as a Function of Discharge - Quality Station Number 16 - Solomon River at Niles

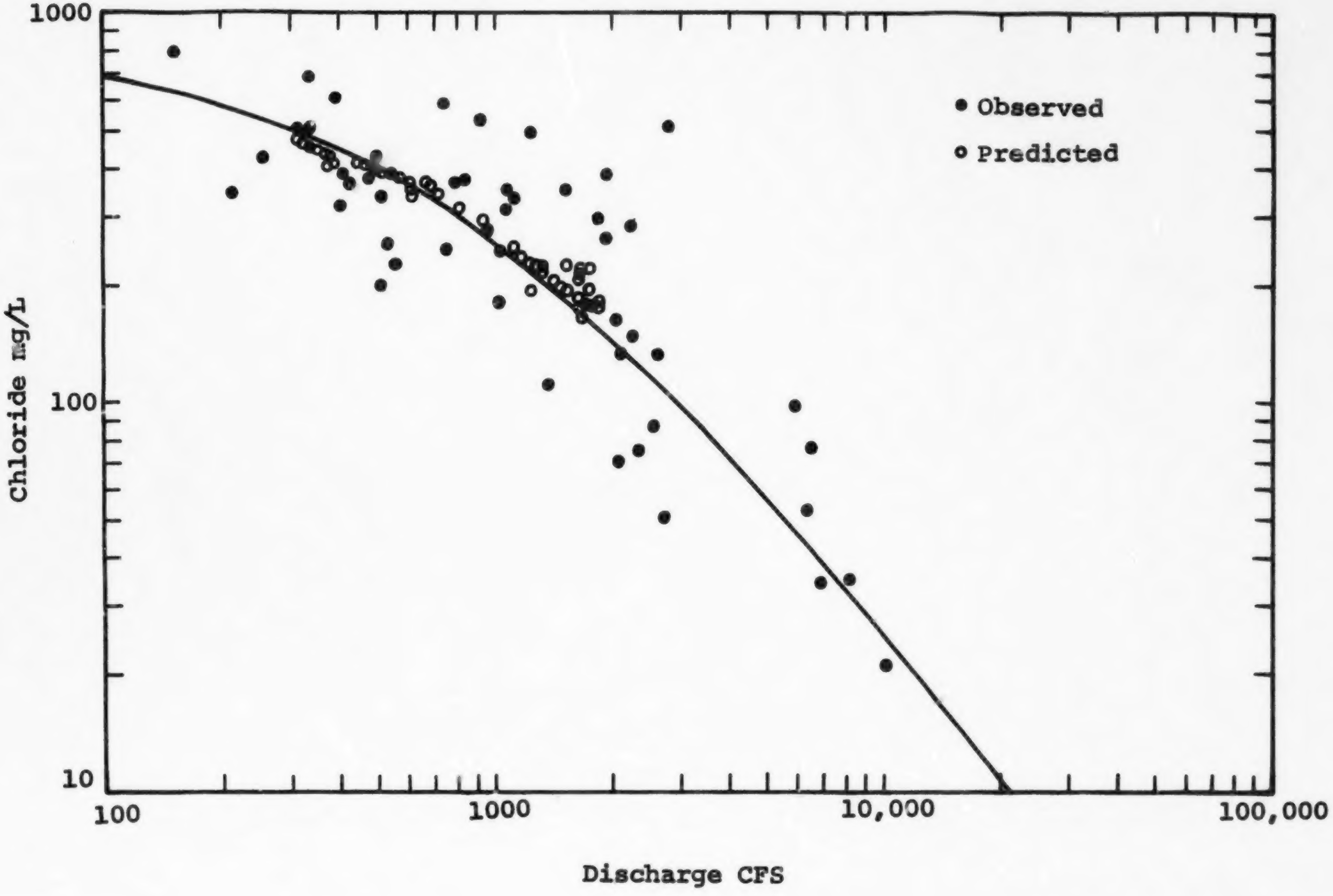


Figure 32 Observed and Predicted Chloride Concentration as a Function of Discharge - Quality Station Number 13 - Smoky Hill River at Enterprise

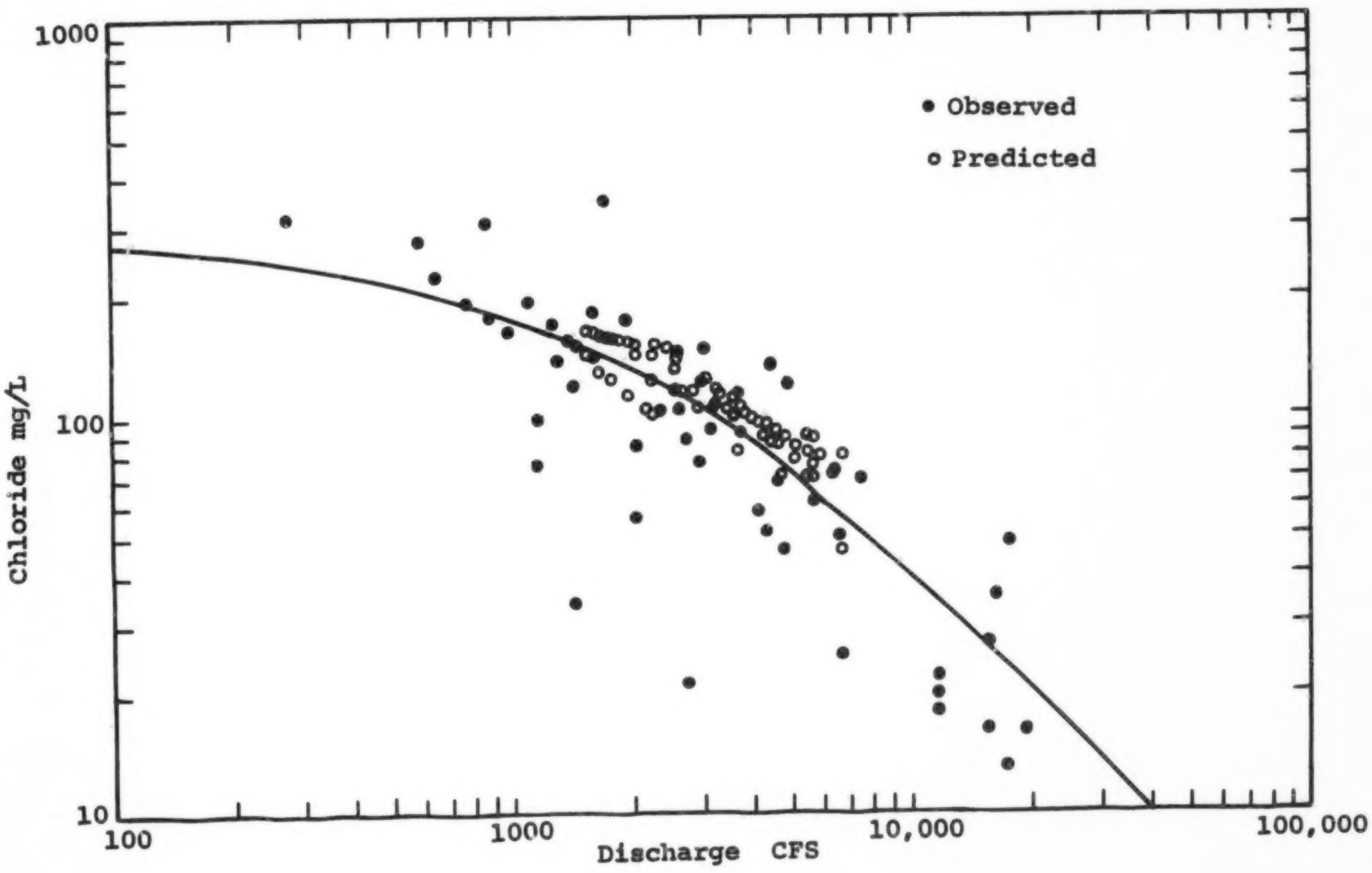


Figure 33 Observed and Predicted Chloride Concentration as a Function of Discharge - Quality Station Number 8 - Kansas River at Wamego

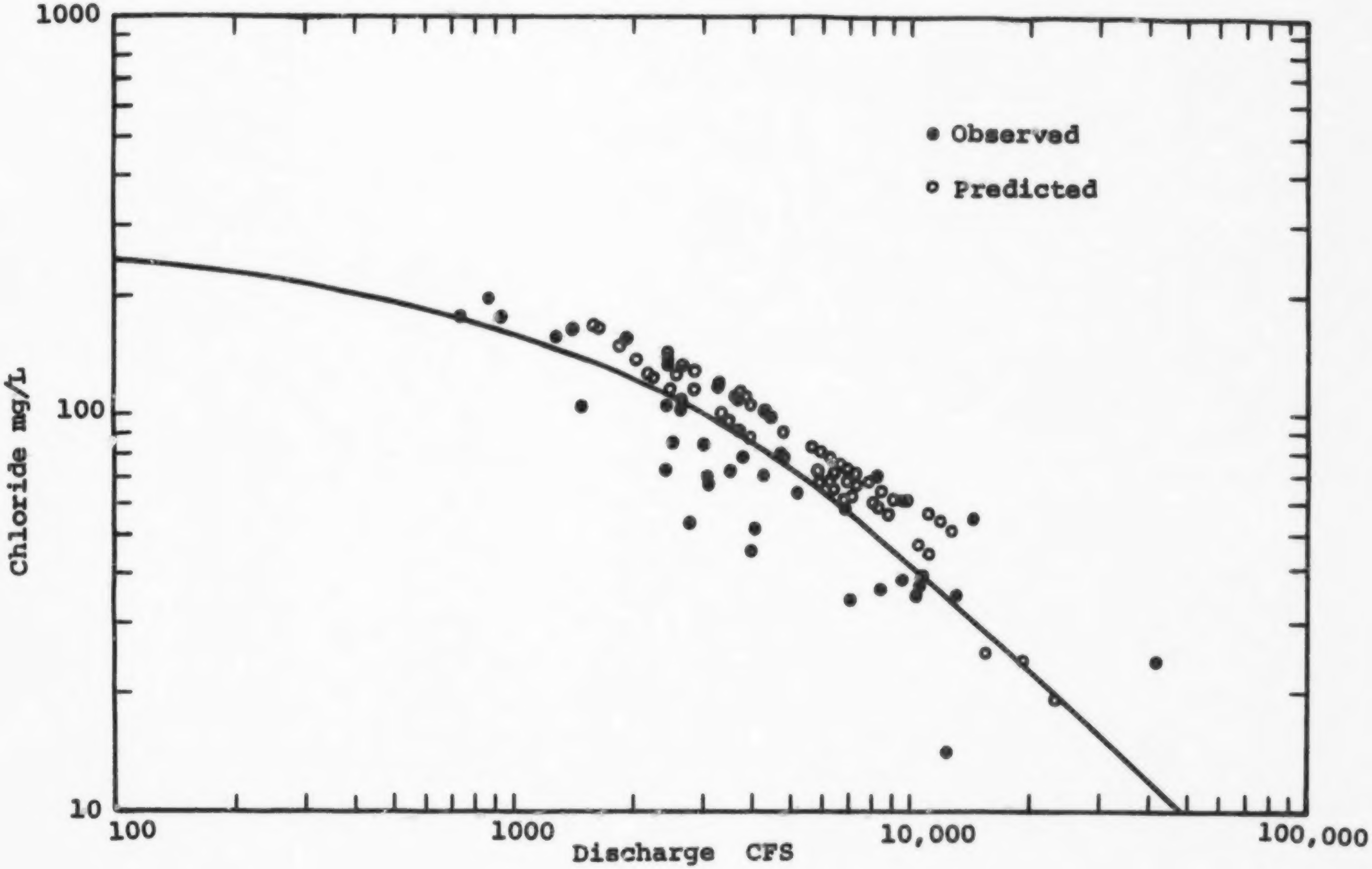


Figure 34 Observed and Predicted Chloride Concentration as a Function of Discharge - Quality Station Number 2 - Kansas River at Bonner Springs

VIII. Discussion

On the basis of the research completed on the Lower Kansas River Model, at the present time the following observations are warranted:

(1) The model will satisfactorily simulate observed discharge records. Additional refinement of the model could be achieved by increasing the number of tributary gaging stations in the western portion of the study area, and by incorporating a bank storage function into the model. Neither refinement appears justified at the present time.

(2) The time-of-flow assumption used in the model produces satisfactory results over the flow range tested. Additional field observations should be conducted to establish the time of flow under extremely low discharge conditions.

(3) The model will satisfactorily simulate the conservative water quality in the basin, but the method currently used to develop the initial conservative vectors and to optimize these vectors is extremely time-consuming. An investigation is being conducted into ways of alleviating this difficulty.

(4) Additional research will be necessary to evaluate the parameters entering the non-conservative water quality model. The most pressing need is to obtain a field evaluation of the reaeration coefficient.

(5) The non-conservative model does not consider the effects of nitrification on the oxygen balance at the present time. Recent work by St. John and O'Connor (24) and by Purdy (25) indicates that the effect of nitrification may be significant under some conditions. This parameter should be reviewed when more information on nitrification is available.

(6) The method used for approximating the relationship between BOD load and agricultural runoff in the model is undoubtedly an oversimplification. Additional research should be directed toward completely defining the problem.

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APPENDIX A

Quasi-Steady-State Model
Fortran Source List

QUASI STEADY STATE MODEL

FORTRAN SOURCE LIST

```
      DIMENSION S(226),QAA(226),QAB(226),HA(226),HB(226),VA(226),VB(226)
      INTEGER SKIP(226),UR1(226),UR2(226),IG1(226),IG2(226)
      REAL MORGA,MK1,MK3,MA,MP,NLIM,CLIM,LA,K2,K1(226),K3(226),A(226),P(
     1226),LO(226),LOC(226),L(226),LTF
      DIMENSION NP1(6),QCRIT(12),E(12),B(12),GAGE(50),Q(226),DMAX(226),
     1CONC(226),C(226),CAA(226),CAB(226),CAC(226),CAD(226),CAE(226),CAF(
     2226),QADC(226),QADC2(226),DO(226),ORGA(226)
C SKIP = ONE FOR EXISTANCE AND ZERO FOR NON-EXISTENCE OF STREAM REACH NUMBER.
      READ(5,20) SKIP
   20 FORMAT(80I1)
C INPUT AND OUTPUT OF CONSTANT VECTOR.
      WRITE(6,886)
  886 FORMAT(1H1//55X,15HCONSTANT VECTOR)
      WRITE(6,21)
   21 FORMAT(1H //10X,116H REACH        UR1        UR2   LENGTH       HA
     1   HB          VA          VB          QAA       QAB          IG1
     2 IG2/)
      DO 30 I=1,226
      IF(SKIP(I).LE.0) GO TO 30
      READ(5,40)UR1(I),UR2(I),L(I),HA(I),HB(I),VA(I),VB(I),QAA(I),QAB(I)
     1,IG1(I),IG2(I),S(I)
   40 FORMAT(2I3,F5.1,6F7.3,2I3,F6.3)
      WRITE(6,41) I,UR1(I),UR2(I),L(I),HA(I),HB(I),VA(I),VB(I),QAA(I),
     1QAB(I),IG1(I),IG2(I)
   41 FORMAT(5X,3I10,F10.2,6F10.3,2I10)
   30 CONTINUE
      WRITE(6,333)
  333 FORMAT(1H1//53X,19HCONSERVATIVE VECTOR)
      WRITE(6,22)
   22 FORMAT(1H0//10X,108H REACH      UR1      UR2  LENGTH    QADC
     1CAA       CAB        CAC        CAD        QADC2       CAE          CAF/)
      DO 31 I=1,226
      IF(SKIP(I).LE.0) GO TO 31
      READ(5,42) QADC(I),CAA(I),CAB(I),CAC(I),CAD(I),QADC2(I),CAE(I),CAF
     1(I)
   42 FORMAT(4F10.0,F7.0,3F8.0)
      WRITE(6,43) I,UR1(I),UR2(I),L(I),QADC(I),CAA(I),CAB(I),CAC(I),CAD(
     1I),QADC2(I),CAE(I),CAF(I)
   43 FORMAT(5X,I10,2I8,F8.2,F10.0,4F10.3,F10.0,2F10.3)
   31 CONTINUE
C     ORGANIC VECTOR
      WRITE (6,121)
  121 FORMAT(1H1,20X,14HORGANIC VECTOR//8H    ORGA,8X,2HK1,8X,2HK3,8X,1H
     1A,9X,1HP/)
      DO 130 I = 1,240
      IF(SKIP(I).EQ. 0) GO TO 130
      READ (5,140) ORGAUI), K1(I), K3(I), A(I), P(I)
  140 FORMAT (1F7.2, 4F7.3)
      WRITE (6,141) ORGA(I), K1(I), K3(I), A(I), P(I)
  141 FORMAT (1H 5F10.3 )
  130 CONTINUE
C     ORGANIC VECTOR IS NOW STORED
  131 READ (5,132) SAT
  132 FORMAT (1F10.2)
      IF(SAT.LE.0.0) CALL EXIT
      WRITE(6,341)
  341 FORMAT(1H1//7H   FLOW,5X,3HNO./)
      DO 330 I = 1,32
      READ (5,340) GAGE(I)
  340 FORMAT (1F9.2)
      WRITE (6,400) GAGE(I), I
  400 FORMAT(1H ,F12.2,I5)
  330 CONTINUE
      READ(5,49)NLIM,CLIM
   49 FORMAT(2F5.2)
      READ(5,500) MORGA,MK1,MK3,MA,MP
  500 FORMAT(5F5.2)
      WRITE (6,506)
  506 FORMAT(24H0ORGA  K1   K3   A    P )
      WRITE(6,501) MORGA,MK1,MK3,MA,MP
  501 FORMAT(1H ,5F5.2////)
      DATA NPT/226,214,180,131,96,41/
      GAGE(1) = 0.0
      Q(1)=0.0
      NNN=1
      DO 2010 I=1,226
      IF(QADC2(I).LE.0.0)QADC2(I)=100000.
 2010 CONTINUE
C     BEGIN SIMULATION FROM UPPERMOST REACH
      DO 60 LK = 1,240
      I = 241-LK
      IF(SKIP(I).EQ.0) GO TO 60
      JR1=UR1(I)
      JR2=UR2(I)
      JG1 = IG1(I)
      JG2 = IG2(I)
      ZORGA=MORGA*ORGA(I)
      ZK1=MK1*K1(I)
      ZK3=MK3*K3(I)
      ZA=MA*A(I)
      ZP=MP*P(I)
      QADD=QAA(I)*(GAGE(JG1)+GAGE(JG2))+QAB(I)
      Q(I)=QADD+Q(JR1)+Q(JR2)
      H=HA(I)*Q(I)**HB(I)
      V=VA(I)*Q(I)**VB(I)
      TF=L(I)/(16.4*V)
      ZORGA=ZORGA+150.*QADD**0.4
      LA=(LO(JR1)+LO(JR2)+ZORGA)/(5.4*Q(I))
      DA=SAT-(DO(JR1)+DO(JR2)+QADD*SAT*5.4)/(5.4*Q(I))
 9000 CALL AK2 ( V, H, S(I), K2 )
      CALL DEFT (ZK1, K2, ZK3, LA, ZP, ZA, DA, TF, DTF )
      DO(I)=(SAT-DTF)*5.4*Q(I)
      CALL LOADT (ZK1, K2, ZK3, LA, ZP, DA, TF, LTF )
      LO(I)=LTF*5.4*Q(I)
      CALL TCR (ZK1, K2, ZK3, LA, ZP,DA, ZA, TC )
      IF(TC.LT.TF) GO TO 70
      DMAX(I)=DTF
      LOC(I)=V*TF*16.4
      GO TO 80
   70 CALL DEFT (ZK1, K2, ZK3, LA, ZP, ZA, DA, TC, DTC )
      DMAX(I)=DTC
      LOC(I)=V*TC*16.4
   80 CONTINUE
      DCONC=0.0
      IF(I.LT.NPT(6))GO TO 51
      IF(I.GT.NPT(NNN))GO TO 51
      READ(5,610) NUM
  610 FORMAT(I10)
      DO 601 J=1,NUM
      READ(5,602)QCRIT(J),E(J),B(J)
  602 FORMAT(F10.0,F20.0,F10.0)
  601 CONTINUE
   51 CONTINUE
      IF(I.GT.NPT(NNN))GO TO 600
      NNN=NNN+1
      NM=NUM+1
      DO 608 K=2,NM
      CADD=E(K-1)*Q(I)**B(K-1)
      IF(QCRIT(K).GT.Q(I)) GO TO 2001
  608 CONTINUE
  600 IF(QADD.GE.QADC(I))GO TO 2000
      CADD=CAA(I)*(QADD**CAB(I))
      GO TO 2001
 2000 IF(QADD.GE.QADC2(I)) GO TO 2002
      CADD=CAC(I)*(QADD**CAD(I))
      GO TO 2001
 2002 CADD=CAE(I)*(QADD**CAF(I))
 2001 C(I)=CADD+C(JR1)+C(JR2)
      CONC(I)=C(I)/((0.0027)*Q(I))
   60 CONTINUE
C   PRINTOUT
```

```
      WRITE(6,991)
  991 FORMAT(1H1,20X,12HCONSERVATIVE//)
      DO 990 I=1,226
      IF(SKIP(I).LE.0)GO TO 990
      IF(CONC(I).LE.CLIM) GO TO 990
      WRITE(6,509) I,CONC(I)
  509 FORMAT(1H ,19H CRITICAL REACH NO.,I4,15H       MAX CONC=,F8.0)
  990 CONTINUE
      WRITE(6,993)
  993 FORMAT(1H1,20X,15HNONCONSERVATIVE//)
      DO 992 I=1,226
      IF(SKIP(I).LE.0) GO TO 992
      IF(DMAX(I).LE.NLIM) GO TO 993
      WRITE(6,508)I,DMAX(I),LOC(I)
  508 FORMAT(1H ,19H CRITICAL REACH NO.,I4,9H    DMAX=,F9.2,8H    LOC=,F
     15.1)
  992 CONTINUE
      GO TO 131
      END
```

APPENDIX B

Dynamic Model for Conservative Pollutants
Fortran Source List

DYNAMIC MODEL FOR CONSERVATIVE POLLUTANTS

FORTRAN SOURCE LIST

```
  1          INTEGER SKIP(226),UR1(226),UR2(226),IG1(226),IG2(226)
  2          DIMENSION S(226),QAA(226),QAB(226),HA(226),HB(226),VA(226),VB(226)
            1,QOUT(226,20),GAGE(35,20),CAA(226),QADC(226),CAB(226),CAC(226),CAD
            2(226),COUT(226,20),QADC2(226),CAE(226),CAF(226)
  3          DIMENSION NPT(6),QCRIT(12),A(12),B(12)
  4          REAL NT,NTTF,L(226),LIMIT
       C ND = NUMBER OF DAYS OF STREAMFLOW
       C IDATE1,IDATE2 = DATE OF FIRST DAY OF STREAMFLOW
  5       10 READ(5,27)ND,IDATE1,IDATE2
 11       27 FORMAT(I10,2A6)
 12          IF(ND.LE.0)  CALL EXIT
       C SKIP = ONE FOR EXISTANCE AND ZERO FOR NON-EXISTENCE OF STREAM REACH NUMBER.
 15          READ(5,20) SKIP
 17       20 FORMAT(80I1)
       C  INPUT OF GAGE FLOW
 20          DO 29 J=1,32
 21          READ(5,28) (GAGE(J,N),N=1,ND)
 26       28 FORMAT(9F8.0)
 27       29 CONTINUE
       C INPUT AND OUTPUT OF CONSTANT VECTOR.
 31          WRITE(6,886)
 32      886 FORMAT(1H1//55X,15HCONSTANT VECTOR)
 33          WRITE(6,21)
 34       21 FORMAT(1H //10X,116H REACH        UR1         UR2   LENGTH       HA
            1    HB           VA          VB          QAA           QAB       IG1
            2 IG2/)
 35          DO 30 I=1,226
 36          IF(SKIP(I).LE.0) GO TO 30
 41          READ(5,40)UR1(I),UR2(I),L(I),HA(I),HB(I),VA(I),VB(I),QAA(I),QAB(I)
            1,IG1(I),IG2(I),S(I)
 42       40 FORMAT(2I3,F5.1,6F7.3,2I3,F6.3)
 43          WRITE(6,41) I,UR1(I),UR2(I),L(I),HA(I),HB(I),VA(I),VB(I),QAA(I),
            1QAB(I),IG1(I),IG2(I)
 44       41 FORMAT(5X,3I10,F10.2,6F10.3,2I10)
 45          DO 25 J=1,ND
 46          COUT(I,J)=0.0
 47      25 QOUT(I,J)=0.0
 51      30 CONTINUE
 53         WRITE(6,333)
 54     333 FORMAT(1H1//53X,19HCONSERVATIVE VECTOR)
 55         WRITE(6,22)
 56      22 FORMAT(1H0//10X,108H REACH      UR1      UR2  LENGTH   QADC
          1CAA        CAB         CAC         CAD         QADC2        CAE          CAF/)
 57         DO 31 I=1,226
 60         IF(SKIP(I).LE.0) GO TO 31
 63         READ(5,42) QADC(I),CAA(I),CAB(I),CAC(I),CAD(I),QADC2(I),CAE(I),CAF
          1(I)
 64      42 FORMAT(4F10.0,F7.0,3F8.0)
 65         WRITE(6,43) I,UR1(I),UR2(I),L(I),QADC(I),CAA(I),CAB(I),CAC(I),CAD(
          1I),QADC2(I),CAE(I),CAF(I)
 66      43 FORMAT(5X,I10,2I8,F8.2,F10.0,4F10.3,F10.0,2F10.3)
 67      31 CONTINUE
 71         DO 37 J=1,ND
 72      37 GAGE(1,J)=0.0
 74         DATA NPT/226,214,180,131,96,41/
 75         NNN=1
 76         DO 2010 I=1,226
 77         IF(QADC2(I).LE.0.0)QADC2(I)=100000.
102    2010 CONTINUE
104         WRITE(6,334) ND,IDATE1,IDATE2
105     334 FORMAT(1H1//15X,15H   CONSERVATIVE/10X,I5,18H  DAYS BEGINNING  ,2A
          16//)
106         READ(5,501) LIMIT
107     501 FORMAT(F10.0)
      CALCULATION OF FLOWS IN THE REACHES
110         DO 60 LK=1,226
111         DCONC=0.0
112         I=227-LK
113         IF(SKIP(I).LE.0) GO TO 60
116         JR1=UR1(I)
117         JR2=UR2(I)
120         JG1=IG1(I)
121         JG2=IG2(I)
122         NT=1.0
123         IF(I.LT.NPT(6))GO TO 51
126         IF(I.GT.NPT(NNN))GO TO 51
131         READ(5,610) NUM
```

```
 610 FORMAT(I10)
     DO 601 J=1,NUM
     READ(5,602)QCRIT(J),A(J),B(J)
 602 FORMAT(F10.0,F20.0,F10.0)
 601 CONTINUE
  51 ITME=NT
     QADD = CAA(I)*(GAGE(JG1,ITME)+GAGE(JG2,ITME)) +QAB(I)
     QRCH=QADD+QOUT(JR1,ITME)+QOUT(JR2,ITME)
     IF(QRCH.LT.1.0)  QRCH=1.0
     IF(I.GT.NPT(NNN))GO TO 600
     NNN=NNN+1
     NM=NUM+1
     DO 608 K=2,NM
     CADD=A(K-1)*QRCH**B(K-1)
     IF(QCRIT(K).GT.QRCH) GO TO 2001
 608 CONTINUE
 600 IF(QADD.GE.QADC(I))GO TO 2000
     CADD=CAA(I)*(QADD**CAB(I))
     GO TO 2001
2000 IF(QADD.GE.QADC2(I)) GO TO 2002
     CADD=CAC(I)*(QADD**CAD(I))
     GO TO 2001
2002 CADD=CAE(I)*(QADD**CAF(I))
2001 CRCH=CADD+COUT(JR1,ITME)+COUT(JR2,ITME)
     V=VA(I)*QRCH**VB(I)
     TF=L(I)/(16.4*V)
     NTTF= NT+TF
     INTTF=NTTF
     IF(INTTF.GT.ND)  GO TO 55
     IINT=NT
     IF(INTTF.GT.IINT)  GO TO 53
     QOUT(I,INTTF)=QOUT(I,INTTF)+QRCH*TF
     COUT(I,INTTF)=COUT(I,INTTF)+CRCH*TF
     GO TO 56
  53 DTF=FLOAT(INTTF)-NT
     IF(DTF.GT.1.0)  GO TO 54
     F1=FLOAT(INTTF)-NT
     QOUT(I,IINT)=QOUT(I,IINT)+QRCH*F1
     COUT(I,IINT)=COUT(I,IINT)+CRCH*F1
     F1=NTTF-FLOAT(INTTF)
     QOUT(I,INTTF)=QOUT(I,INTTF)+QRCH*F1
     COUT(I,INTTF)=COUT(I,INTTF)+CRCH*F1
     GO TO 56
```

```
   54 F1=FLOAT(IINT+1)-NT
      QOUT(I,IINT)=QOUT(I,IINT)+QRCH*F1
      COUT(I,IINT)=COUT(I,IINT)+CRCH*F1
      ICT=1
   58 INCT=IINT+ICT
      F1=NTTF-FLOAT(INCT)
      IF(F1.GT.1.0)  GO TO 52
      QOUT(I,INCT)=QOUT(I,INCT)+QRCH*F1
      COUT(I,INCT)=COUT(I,INCT)+CRCH*F1
      GO TO 56
   52 QOUT(I,INCT)=QOUT(I,INCT)*QRCH
      COUT(I,INCT)=COUT(I,INCT)+CRCH
      ICT=ICT+1
      GO TO 58
   56 NT=NTTF
      GO TO 51
   55 JN=ND-2
      DO 502 K=2,JN
      CONC=COUT(I,K)/((0.0027)*QOUT(I,K))
      IF(CONC.LE.DCONC) GO TO 502
      DCONC=CONC
      JDAY=K
  502 CONTINUE
      IF(DCONC.LT.LIMIT) GO TO 60
      WRITE(6,500) I,DCONC,JDAY
  500 FORMAT(1H ,19H CRITICAL REACH NO.,I4,10H MAX CONC=,F8.0,6H  DAY=,I
     14)
   60 CONTINUE
      GO TO 10
      END
```

APPENDIX C

Dynamic Model for Non-Conservative Pollutants
Fortran Source List

DYNAMIC MODEL FOR NON CONSERVATIVE POLLUTANTS

FORTRAN SOURCE LIST

```
1           INTEGER SKIP(226),UR1(226),UR2(226),IG1(226),IG2(226)
2           DIMENSION DO(226,20),ORGA(226),NPT(6)
3           DIMENSION S(226),QAA(226),QAB(226),HA(226),HB(226),VA(226),VB(226)
           1,QOUT(226,20),GAGE(35,20)
4           REAL NT,NTTF,L(226),LO(226,20),LRCH
5           REAL MORGA,MK1,MK3,MA,MP,LIMIT,LTF,LA,K2,K1(226),K3(226),A(226),P(
           1226)
      C ND = NUMBER OF DAYS OF STREAMFLOW
      C IDATE1,IDATE2 = DATE OF FIRST DAY OF STREAMFLOW
6        10 READ(5,27)ND,IDATE1,IDATE2
12       27 FORMAT(I10,2A6)
13          IF(ND.LE.0)  CALL EXIT
      C SKIP = ONE FOR EXISTANCE AND ZERO FOR NON-EXISTENCE OF STREAM REACH NUMBER.
16          READ(5,20) SKIP
20       20 FORMAT(80I1)
21          IP=4
      C INPUT OF GAGE FLOW
22          DO 29 J=1,32
23          READ(5,28) (GAGE(J,N),N=1,ND)
30       28 FORMAT(9F8.0)
31       29 CONTINUE
      C INPUT AND OUTPUT OF CONSTANT VECTOR.
33          WRITE(6,886)
34      886 FORMAT(1H1//55X,15HCONSTANT VECTOR)
35          WRITE(6,21)
36       21 FORMAT(1H //10X,116H REACH        UR1        UR2   LENGTH       HA
           1     HB          VA          VB          QAA        QAB             IG1
           2 IG2/)
37          DO 30 I=1,226
40          IF(SKIP(I).LE.0) GO TO 30
43          READ(5,40)UR1(I),UR2(I),L(I),HA(I),HB(I),VA(I),VB(I),QAA(I),QAB(I)
           1,IG1(I),IG2(I),S(I)
44       40 FORMAT(2I3,F5.1,6F7.3,2I3,F6.3)
45          WRITE(6,41) I,UR1(I),UR2(I),L(I),HA(I),HB(I),VA(I),VB(I),QAA(I),
           1QAB(I),IG1(I),IG2(I)
46       41 FORMAT(5X,3I10,F10.2,6F10.3,2I10)
```

I

```
      DO 25 J=1,ND
      DO(I,J)=0.0
      LO(I,J)=0.0
   25 QOUT(I,J)=0.0
   30 CONTINUE
C     NONCONSERVATIVE VECTOR
      WRITE (6,121)
  1210FORMAT(25H1  NONCONSERVATIVE VECTOR/
     149H     ORGA          K1          K3           A          P)
      DO 130 I = 1,226
      IF(SKIP(I).EQ. 0) GO TO 130
      READ (5,140) ORGA(I), K1(I), K3(I), A(I), P(I)
  140 FORMAT (1F7.2, 4F7.3)
      WRITE (6,141) ORGA(I), K1(I), K3(I), A(I), P(I)
  141 FORMAT (1H 5F10.3 )
  130 CONTINUE
      READ (5,132) SAT
  132 FORMAT (1F10.2)
C     ORGANIC VECTOR IS NOW STORED
      READ(5,49) LIMIT
   49 FORMAT (F5.2)
      READ(5,500) MORGA,MK1,MK3,MA,MP
  500 FORMAT(5F5.2)
      WRITE (6,506)
  506 FORMAT(24H ORGA  K1   K3   A    P )
      WRITE(6,501) MORGA,MK1,MK3,MA,MP
  501 FORMAT(1H ,5F5.2////)
      DO 37 J=1,ND
   37 GAGE(1,J)=0.0
      DATA NPT/226,214,180,131,96,41/
      WRITE(6,333) ND,IDATE1,IDATE2
  333 FORMAT(1H1//15X,15HNONCONSERVATIVE/10X,I5,18H  DAYS BEGINNING  ,2A
     16//)
CALCULATION OF FLOWS IN THE REACHES
      DO 60 LK=1,226
      I=227-LK
      IF(SKIP(I).LE.0) GO TO 60
      JR1=UR1(I)
      JR2=UR2(I)
      JG1=IG1(I)
      JG2=IG2(I)
      ADMAX=0.0
      NT=1.0
      ZORGA=MORGA*ORGA(I)
      ZK1=MK1*K1(I)
      ZK3=MK3*K3(I)
      ZA=MA*A(I)
      ZP=MP*P(I)
      KMK=0
   51 ITME=NT
      QADD = QAA(I)*(GAGE(JG1,ITME)+GAGE(JG2,ITME)) +QAB(I)
      JMJ=QADD
      QRCH=QADD+QOUT(JR1,ITME)+QOUT(JR2,ITME)
      IF(QRCH.LT.1.0)  QRCH=1.0
      H=HA(I)*QRCH**HB(I)
      V=VA(I)*QRCH**VB(I)
      TF=L(I)/(16.4*V)
      IF(KMK.EQ.JMJ)GO TO 8000
      ZORGA=ZORGA+150.*QADD**0.4
      LA=(LO(JR1,ITME)+LO(JR2,ITME)+ZORGA)/(5.4*QRCH)
      DA=SAT-(DO(JR1,ITME)+DO(JR2,ITME)+QADD*SAT*5.4)/(5.4*QRCH)
      KMK=QADD
 8000 IF(DA.LT.0.0)DA=0.0
 9000 CALL AK2 ( V, H, S(I), K2 )
 9001 CONTINUE
      CALL DEFT (ZK1, K2, ZK3, LA, ZP, ZA, DA, TF, DTF )
      DRCH=(SAT-DTF)*5,4*QRCH
      CALL LOADT (ZK1, K2, ZK3, LA, ZP, DA, TF, LTF )
      LRCH=LTF*5.4*QRCH
      CALL TCR (ZK1, K2, ZK3, LA, ZP,DA, ZA, TF, TC )
      IF(TC.LT.TF) GO TO 70
      NM=ND-2
      IF(ITME.GE.NM) GO TO 80
      DMAX=DTF
      LOC=TF*V*16.4
      GO TO 80
   70 CALL DEFT (ZK1, K2, ZK3, LA, ZP, ZA, DA, TC, DTC )
      IF(ITME.GE.NM) GO TO 80
      DMAX=DTC
      LOC=V*TC * 16.4
   80 CONTINUE
      IF(DMAX.LT.ADMAX) GO TO 81
      ADMAX=DMAX
      IDAY=ITME
   81 NTTF= NT+TF
      INTTF=NTTF
      IF(INTTF.GT.ND)  GO TO 55
      IINT=NT
      IF(INTTF.GT.IINT)  GO TO 53
      QOUT(I,INTTF)=QOUT(I,INTTF)+QRCH*TF
      DO(I,INTTF)=DO(I,INTTF)+DRCH*TF
      LO(I,INTTF)=LO(I,INTTF)+LRCH*TF
      GO TO 56
   53 DTF=FLOAT(INTTF)-NT
      IF(DTF.GT.1.0)  GO TO 54
      F1=FLOAT(INTTF)-NT
      QOUT(I,IINT)=QOUT(I,IINT)+QRCH*F1
      DO(I,IINT)=DO(I,IINT)+DRCH*F1
      LO(I,IINT)=LO(I,IINT)+LRCH*F1
      F1=NTTF-FLOAT(INTTF)
      QOUT(I,INTTF)=QOUT(I,INTTF)+QRCH*F1
      DO(I,INTTF)=DO(I,INTTF)+DRCH*F1
      LO(I,INTTF)=LO(I,INTTF)+LRCH*F1
      GO TO 56
   54 F1=FLOAT(IINT+1)-NT
      QOUT(I,IINT)=QOUT(I,IINT)+QRCH*F1
      DO(I,IINT)=DO(I,IINT)+DRCH*F1
      LO(I,IINT)=LO(I,IINT)+LRCH*F1
      ICT=1
   58 INCT=IINT+ICT
      F1=NTTF-FLOAT(INCT)
      IF(F1.GT.1.0)  GO TO 52
      QOUT(I,INCT)=QOUT(I,INCT)+QRCH*F1
      DO(I,INCT)=DO(I,INCT)+DRCH*F1
      LO(I,INCT)=LO(I,INCT)+LRCH*F1
      GO TO 56
   52 QOUT(I,INCT)=QOUT(I,INCT)+QRCH
      DO(I,INCT)=DO(I,INCT)+DRCH
      LO(I,INCT)=LO(I,INCT)+LRCH
      ICT=ICT+1
      GO TO 58
   56 NT=NTTF
   90 GO TO 51
   55 CONTINUE
```

```
      IF(ADMAX.LT.LIMIT)GO TO 60
      WRITE(6,508)I,ADMAX,LOC,IDAY
  508 FORMAT(1H ,19H CRITICAL REACH NO.,I4,7H  DMAX=,F8.2,6H  LOC=,I5,3X
     1,5H DAY=,I4)
   60 CONTINUE
      GO TO 10
      END
```

APPENDIX D

Subroutines for the Non-Conservative Model
Fortran Source List

SUBROUTINES FOR THE NON CONSERVATIVE MODEL

FORTRAN SOURCE LIST

```
      SUBROUTINE LOADT (K1, K2, K3, LA, P, DA, T, LT )
      REAL K1,K2,K3,LA,P,DA,T,LT,K
      K=K1+K3
      LT=(LA-(P/K))*(EXP(-K*T))+(P/K)
      RETURN
      END
```

```
      SUBROUTINE AK2 (V100, H100, S, ZKE )
      REAL  KE
      T.= 20.0
      F=V100/SQRT(32.2*H100)
      E=30.0*S*V100
      TX=0.0
      IF(T.NE.30.0) TX=(ABS(30.0-T))**1.5
      B=0.976+0.0137*TX
      A=9.68+0.054*(T-20.0)
      C4=0.9+F
      CA=1.0+F*F
      KE=.12*CA*A*E**.375*(1.0/TANH(B*E**.125/C4**.5))/(C4**1.5*H100)
      ZKE=KE*2.303
      RETURN
      END
```

I

```
      SUBROUTINE DEFT (K1, K2, K3, LA, P, A, DA, T, DT )
      REAL K1,K2,K3,K,LA,P,A,DA,T,DT
      K=K1+K3
     0DT= (K1/(K2-K))*((LA-(P/K))*(EXP(-K*T)-EXP(-K2*T)))+((K1/K2)*((P/K
     2)-(A/K1))* (1.0-EXP(-K2*T)))  +DA*EXP(-K2*T)
      RETURN
      END
```

```
      SUBROUTINE TCR (K1,   K2, K3, LA, P, DA, A, TF, TC )
      REAL K1,K2,K3,LA,P,DA,A.TC  ,K
      K=K1+K3
      ZLOG=(K2/K)+((K2-K)/((K*LA)-P))*((P/K)-((A+K2*DA)/K1))
      IF(ZLOG.LE.0.0) GO TO 10
     0TC=(1.0/(K2-K))*ALOG((K2/K)+((K2-K)/((K*LA)-P))*((P/K)-((A+K2*DA)
     1/K1)) )
      GO TO 11
   10 TC = TF
   11 RETURN
      END
```

APPENDIX E

Constant Vectors

Errata

The following corrections should be applied to the designated constants in the constant vector compilation.

Reach 15	HA should be 0.430
Reach 76	HB should be 0.476
Reach 176	VB should be 0.265

CONSTANT VECTOR

REACH	UR1	UR2	LENGTH	HA	HB	VA	VB	QAA	QAB	IG1	IG2
3	4	5	16.00	0.139	0.384	0.139	0.325	0.262	0.000	3	1
4	1	1	11.00	2.000	0.000	1.000	0.000	0.184	0.000	3	1
5	7	1	5.00	0.139	0.384	0.139	0.325	0.056	0.000	3	1
7	10	1	10.00	0.139	0.384	0.139	0.325	0.268	10.000	3	2
10	15	20	3.00	0.139	0.384	0.139	0.325	0.014	3.000	3	2
15	16	1	17.00	4.430	0.362	0.500	0.189	0.384	0.000	2	1
16	1	1	5.00	0.430	0.362	0.500	0.189	1.000	0.000	2	1
20	25	29	7.00	0.139	0.384	0.139	0.325	0.104	7.000	3	2
25	26	1	13.00	0.320	0.438	0.550	0.190	0.284	0.000	3	1
26	1	1	10.00	0.320	0.438	0.550	0.190	1.000	0.000	3	1
29	30	1	10.00	0.139	0.384	0.139	0.325	0.103	10.000	3	2
30	35	1	13.00	0.013	0.640	0.368	0.210	0.136	13.000	3	2
35	40	45	1.00	0.013	0.640	0.368	0.210	0.000	0.000	1	1
40	41	1	31.00	0.100	0.508	0.380	0.270	0.584	0.000	2	1
41	1	1	20.00	0.100	0.508	0.380	0.270	1.000	0.000	5	1
45	50	55	17.00	0.013	0.640	0.368	0.210	0.301	0.000	3	6
50	51	1	7.00	0.220	0.457	0.670	0.206	0.145	0.000	6	1
51	1	1	5.00	0.220	0.457	0.670	0.206	1.000	0.000	6	1
55	59	1	2.00	0.194	0.310	0.170	0.300	0.000	0.000	1	1
59	60	64	18.00	0.194	0.310	0.170	0.300	0.786	0.000	6	1
60	61	1	16.00	0.150	0.469	0.450	0.285	0.291	0.000	8	1
61	62	1	15.40	0.150	0.469	0.450	0.285	0.228	0.000	8	1
62	1	1	5.00	0.150	0.469	0.450	0.285	1.000	0.000	8	1
64	65	1	8.00	0.194	0.310	0.170	0.300	0.139	0.000	8	1
65	70	85	10.00	0.194	0.310	0.170	0.300	0.359	0.000	6	1
70	75	80	18.00	0.220	0.485	0.740	0.162	0.033	0.000	10	1
75	76	1	10.00	0.250	0.476	0.660	0.206	0.438	0.000	9	1
76	1	1	5.00	0.250	0.467	0.660	0.206	1.000	0.000	9	1
80	81	1	10.00	0.220	0.485	0.740	0.162	0.337	0.000	10	1
81	1	1	10.00	0.220	0.485	0.740	0.162	1.000	0.000	10	1
85	90	1	8.00	0.055	0.490	0.333	0.239	0.054	0.000	8	1
90	95	115	18.00	0.055	0.490	0.333	0.239	0.446	0.000	8	1
95	96	1	7.30	0.078	0.550	0.290	0.276	0.139	0.000	8	1
96	1	1	9.70	0.078	0.550	0.290	0.276	1.000	0.000	12	1
115	120	125	18.50	0.120	0.420	0.270	0.284	0.794	0.000	8	1
120	121	1	14.60	0.123	0.500	0.370	0.300	0.273	0.000	15	1
121	1	1	5.00	0.123	0.500	0.370	0.300	1.000	0.000	15	1
125	129	135	4.00	0.120	0.420	0.270	0.284	0.155	0.000	15	1
129	130	1	11.10	0.064	0.500	0.300	0.266	0.074	0.000	17	1
130	131	1	7.50	0.030	0.603	0.385	0.234	0.167	0.000	18	1
131	1	1	5.00	0.030	0.603	0.385	0.234	1.000	0.000	16	1
135	140	145	9.00	0.100	0.513	0.063	0.500	0.109	0.000	17	1
140	141	1	10.00	0.220	0.495	0.430	0.254	0.230	0.000	17	1
141	142	1	51.00	0.220	0.495	0.430	0.254	0.917	0.000	17	1
142	1	1	5.00	0.220	0.495	0.430	0.254	0.083	0.000	17	1
145	150	154	14.80	0.100	0.513	0.063	0.500	0.209	0.000	17	1
150	151	1	6.00	0.560	0.340	0.210	0.280	0.100	0.000	18	1
151	1	1	5.00	0.560	0.340	0.210	0.280	1.000	0.000	18	1
154	155	1	18.40	0.100	0.513	0.063	0.500	0.132	0.000	17	18
155	160	165	10.00	0.100	0.513	0.063	0.500	0.447	0.000	18	1
160	161	1	12.00	0.470	0.358	0.500	0.193	0.168	0.000	20	1
161	1	1	10.00	0.470	0.358	0.500	0.193	1.000	0.000	20	1
165	170	185	16.00	0.100	0.513	0.063	0.500	1.098	0.000	20	1
170	171	1	16.90	0.160	0.534	0.300	0.227	0.287	0.000	18	1

115	120	125	10.90	0.120	0.420	0.270	0.254	0.794	0.000	8	1
120	121	1	14.60	0.123	0.500	0.370	0.300	0.273	0.000	15	1
121	1	1	5.00	0.123	0.500	0.370	0.300	1.000	0.000	15	1
125	129	135	4.00	0.120	0.420	0.270	0.264	0.155	0.000	15	1
129	130	1	11.10	0.064	0.500	0.300	0.266	0.074	0.000	17	1
130	131	1	7.50	0.030	0.603	0.385	0.234	0.167	0.000	18	1
131	1	1	5.00	0.030	0.603	0.385	0.234	1.000	0.000	16	1
135	140	145	9.00	0.100	0.513	0.063	0.500	0.109	0.000	17	1
140	141	1	10.00	0.220	0.495	0.430	0.254	0.230	0.000	17	1
141	142	1	51.00	0.220	0.495	0.430	0.254	0.917	0.000	17	1
142	1	1	5.00	0.220	0.495	0.430	0.254	0.083	0.000	17	1
145	150	154	14.80	0.100	0.513	0.063	0.500	0.209	0.000	17	1
150	151	1	6.00	0.560	0.340	0.210	0.280	0.100	0.000	18	1
151	1	1	5.00	0.560	0.340	0.210	0.280	1.000	0.000	18	1
154	155	1	18.40	0.100	0.513	0.063	0.500	0.132	0.000	17	18
155	160	165	10.00	0.100	0.513	0.063	0.500	0.447	0.000	18	1
160	161	1	12.00	0.470	0.358	0.500	0.193	0.168	0.000	20	1
161	1	1	10.00	0.470	0.358	0.500	0.193	1.000	0.000	20	1
165	170	185	16.00	0.100	0.513	0.063	0.500	1.098	0.000	20	1
170	171	1	16.90	0.160	0.534	0.300	0.227	0.287	0.000	18	1
171	172	174	33.10	0.160	0.534	0.300	0.227	0.740	0.000	18	1
172	173	1	10.00	0.310	0.446	0.390	0.263	0.165	0.000	29	1
173	1	1	5.00	0.310	0.446	0.390	0.263	1.000	0.000	29	1
174	175	1	10.00	0.160	0.534	0.300	0.227	0.167	0.000	18	1
175	176	1	28.00	0.160	0.534	0.300	0.227	0.334	0.000	18	29
176	177	1	10.00	0.160	0.512	0.360	0.285	0.255	0.000	29	1
177	180	1	41.00	0.160	0.512	0.360	0.265	0.415	0.000	29	1
180	1	1	28.00	0.160	0.512	0.360	0.265	1.000	0.000	22	1
185	190	200	10.00	0.087	0.550	0.200	0.350	0.066	0.000	23	1
190	191	1	22.70	0.410	0.340	1.110	0.029	1.174	0.000	23	1
191	1	1	10.00	0.410	0.340	1.110	0.029	1.000	0.000	23	1
200	201	1	7.40	0.087	0.550	0.200	0.350	0.066	0.000	23	1
201	205	215	4.60	0.087	0.550	0.200	0.350	0.091	0.000	23	1
205	208	1	54.50	0.500	0.380	0.175	0.360	0.795	0.000	18	1
208	209	1	38.00	0.500	0.380	0.175	0.360	1.618	0.000	30	1
209	210	1	25.00	0.500	0.380	0.175	0.360	1.712	0.000	30	1
210	211	213	5.30	0.500	0.380	0.175	0.360	0.104	0.000	30	1
211	212	1	15.00	0.340	0.460	0.800	0.130	0.804	0.000	30	1
212	211	1	10.00	0.340	0.460	0.800	0.130	1.000	0.000	30	1
213	214	1	12.20	0.150	0.485	0.550	0.200	0.117	0.000	30	1
214	1	1	10.00	0.150	0.485	0.550	0.200	1.000	0.000	32	1
215	216	1	12.00	0.120	0.524	0.500	0.234	0.023	0.000	18	1
216	220	1	17.40	0.120	0.524	0.500	0.234	0.014	0.000	18	1
220	225	1	39.40	0.440	0.507	0.520	0.331	0.255	0.000	31	1
225	226	1	36.30	0.330	0.501	0.880	0.151	0.336	0.000	31	1
226	1	1	2.00	0.330	0.501	0.880	0.151	1.000	0.000	27	1

B

APPENDIX F

Chloride Conservative Vectors

CHLORIDE CONSERVATIVE VECTOR

REACH	UR1	UR2	LENGTH	QADC	CAA	CAB	CAC	CAD
3	4	5	16.00	100000.	2.183	0.410	-0.000	-0.000
4	1	1	11.00	100000.	2.183	0.410	-0.000	-0.000
5	7	1	5.00	100000.	2.183	0.410	-0.000	-0.000
7	10	1	10.00	100000.	2.183	0.410	-0.000	-0.000
10	15	20	3.00	100000.	2.183	0.410	-0.000	-0.000
15	16	1	17.00	100000.	2.183	0.410	-0.000	-0.000
16	1	1	5.00	100000.	2.183	0.410	-0.000	-0.000
20	25	29	7.00	100000.	2.183	0.410	-0.000	-0.000
25	26	1	13.00	100000.	2.183	0.410	-0.000	-0.000
26	1	1	10.00	100000.	2.183	0.410	-0.000	-0.000
29	30	1	10.00	100000.	2.183	0.410	-0.000	-0.000
30	35	1	13.00	100000.	2.183	0.410	-0.000	-0.000
35	40	45	1.00	100000.	9.588	0.139	-0.000	-0.000
40	41	1	31.00	100000.	9.588	0.139	-0.000	-0.000
41	1	1	20.00	100000.	9.588	0.139	-0.000	-0.000
45	50	55	17.00	100000.	9.588	0.139	-0.000	-0.000
50	51	1	7.00	100000.	9.588	0.139	-0.000	-0.000
51	1	1	5.00	100000.	9.588	0.139	-0.000	-0.000
55	59	1	2.00	100000.	9.588	0.139	-0.000	-0.000
59	60	64	18.00	100000.	9.588	0.139	-0.000	-0.000
60	61	1	16.00	100000.	9.588	0.139	-0.000	-0.000
61	62	1	15.40	100000.	9.588	0.139	-0.000	-0.000
62	1	1	5.00	100000.	9.588	0.139	-0.000	-0.000
64	65	1	8.00	100000.	9.588	0.139	-0.000	-0.000
65	70	85	10.00	100000.	9.588	0.139	-0.000	-0.000
70	75	80	18.00	100000.	9.588	0.139	-0.000	-0.000
75	76	1	10.00	100000.	9.588	0.139	-0.000	-0.000
76	1	1	5.00	100000.	9.588	0.139	-0.000	-0.000
80	81	1	10.00	100000.	9.588	0.139	-0.000	-0.000
81	1	1	10.00	100000.	9.588	0.139	-0.000	-0.000
85	90	1	8.00	100000.	9.588	0.139	-0.000	-0.000
90	95	115	18.00	2250.	3.262	0.470	123.000	0.000
95	96	1	7.30	2250.	3.262	0.470	123.000	0.000
96	1	1	9.70	2250.	3.262	0.470	123.000	0.000

REACH	UR1	UR2	LENGTH	QADC	CAA	CAB	CAC	CAD
115	120	125	18.50	2250.	3.262	0.470	123.000	0.000
120	121	1	14.60	2250.	3.262	0.470	123.000	0.000
121	1	1	5.00	2250.	3.262	0.470	123.000	0.000
125	129	135	4.00	2250.	3.262	0.470	123.000	0.000
129	130	1	11.10	2250.	3.262	0.470	123.000	0.000
130	131	1	7.50	2250.	3.262	0.470	123.000	0.000
131	1	1	5.00	2250.	3.262	0.470	123.000	0.000
135	140	145	9.00	2250.	3.262	0.470	123.000	0.000
140	141	1	10.00	2250.	3.262	0.470	123.000	0.000
141	142	1	51.00	2250.	3.262	0.470	123.000	0.000
142	1	1	5.00	2250.	3.262	0.470	123.000	0.000
145	150	154	14.80	2250.	3.262	0.470	123.000	0.000
150	151	1	6.00	2250.	3.262	0.470	123.000	0.000
151	1	1	5.00	2250.	3.262	0.470	123.000	0.000
154	155	1	18.40	2250.	3.262	0.470	123.000	0.000
155	160	165	10.00	165.	2.562	0.747	91.880	0.046
160	161	1	12.00	165.	2.562	0.747	91.880	0.046
161	1	1	10.00	165.	2.562	0.747	91.880	0.046
165	170	185	16.00	165.	2.562	0.747	91.880	0.046
170	171	1	16.90	165.	2.562	0.747	91.880	0.046
171	172	174	33.10	120.	0.796	0.954	78.000	0.000
172	173	1	10.00	120.	0.796	0.954	78.000	0.000
173	1	1	5.00	120.	0.796	0.954	78.000	0.000
174	175	1	10.00	120.	0.796	0.954	78.000	0.000
175	176	1	28.00	120.	0.796	0.954	78.000	0.000
176	177	1	10.00	120.	0.796	0.954	78.000	0.000
177	180	1	41.00	120.	0.796	0.954	78.000	0.000
180	1	1	28.00	120.	0.796	0.954	78.000	0.000
185	190	200	10.00	165.	2.562	0.747	91.880	0.046
190	191	1	22.70	165.	2.562	0.747	91.880	0.046
191	1	1	10.00	165.	2.562	0.747	91.880	0.046
200	201	1	7.40	165.	2.562	0.747	91.880	0.046
201	205	215	4.60	165.	2.562	0.747	91.880	0.046
205	208	1	54.50	165.	2.562	0.747	91.880	0.046
208	209	1	38.00	660.	13.000	0.308	11595.000	-0.740
209	210	1	25.00	660.	13.000	0.308	11595.000	-0.740

II

REACH	UR1	UR2	LENGTH	QADC	CAA	CAB	CAC	CAD
210	211	213	5.30	660.	13.000	0.308	11595.000	-0.740
211	212	1	15.00	660.	13.000	0.308	11595.000	-0.740
212	211	1	10.00	660.	13.000	0.308	11595.000	-0.740
213	214	1	12.20	660.	13.000	0.308	11595.000	-0.740
214	1	1	10.00	660.	13.000	0.308	11595.000	-0.740
215	216	1	12.00	165.	2.562	0.747	91.880	0.046
216	220	1	17.40	165.	2.562	0.747	91.880	0.046
220	225	1	39.40	165.	2.562	0.747	91.880	0.046
225	226	1	36.30	165.	2.562	0.747	91.880	0.046
226	1	1	2.00	165.	2.562	0.747	91.880	0.046

CHLORIDE CONSERVATIVE VECTOR END REACHES

REACH	QCRIT	A	B
41	100000.	0.310	0.589
96	330.	0.112	0.876
96	700.	0.167	0.806
96	2700.	0.269	0.735
96	6500.	0.825	0.594
96	10700.	1.960	0.495
96	50000.	7.339	0.352

REACH	QCRIT	A	B
131	66.	0.207	0.970
131	121.	0.373	0.829
131	216.	0.666	0.711
131	400.	0.947	0.647
131	900.	1.630	0.555
131	1640.	3.328	0.450
131	2200.	10.190	0.299
131	4050.	17.720	0.227
131	6000.	116.000	0.000
131	30000.	1044.000	-0.252
180	80.	0.993	0.756
180	320.	1.650	0.644
180	550.	6.864	0.397
180	940.	84.000	0.000
180	1300.	12150.000	-0.391
180	2450.	8833.000	-0.667
180	10000.	500800.000	-1.184
214	3.	7.672	0.604
214	9.	7.984	0.548
214	30.	9.639	0.467
214	60.	11.210	0.423
214	132.	16.180	0.334
214	275.	25.130	0.243
214	450.	55.630	0.101
214	770.	103.000	0.000
214	1350.	306.500	-0.163
214	2100.	877.100	-0.309
214	10000.	2583.000	-0.451
226	80.	0.710	0.980
226	220.	1.011	0.900
226	370.	5.504	0.585
226	580.	25.710	0.324
226	1550.	200.000	0.000
226	2500.	1351000.000	-1.199
226	10000.	*************	-2.003

APPENDIX G

Summary of Computer Program Symbols

I

Summary of Computer Program Symbols

A - Rate of removal of oxygen along reach.

ADMAX - Highest value of maximum oxygen deficit during period.

AK2 - Identifier of subroutine to compute K2.

B - Exponent for CADD function from reservoirs.

CAA - Coefficient for CADD function.

CAB - Exponent for CADD function.

CAC - Coefficient for CADD function.

CAD - Exponent for CADD function.

CADD - Tons per day of a conservative pollutant added locally.

CAE - Coefficient for CADD function.

CAF - Exponent for CADD function.

CLIM - Conservative limit.

CONC - Concentration in mg./l. of a conservative pollutant.

COUT - Tons per day of a conservative pollutant leaving a reach.

CRCH - Tons per day of a conservative pollutant entering a reach.

DA - Initial oxygen deficit in mg./l.

DCONC - Highest concentration during period.

DEFT - Identifier of subroutine to compute oxygen deficit.

DMAX - Maximum oxygen deficit in the reach in mg./l.

DO - Dissolved oxygen leaving reach in lbs./day.

DRCH - Dissolved oxygen in reach in lbs./day.

DT - Oxygen deficit after T days.

DTC - Oxygen deficit at critical point of sag.

DTF - Oxygen deficit at outlet of reach.

E - Coefficient for CADD function from reservoirs.

F1 - Partial-day factor in dynamic model.

GAGE - Boundary condition stream-gage reading in cfs.

H - Hydraulic depth in reach in feet.

HA - Coefficient for H function.

HB - Exponent for H function.
I - Integer identifier for reach.
ICT - Multiple-day integer for dynamic model.
IDATE1 - Field identifier for date of historic period.
IDATE2 - Field identifier for date of historic period.
IG1 - Integer identifier of stream gage for QADD function.
IG2 - Integer identifier of stream gage for QADD function.
IINT - Integer truncate of day of the period plus TF.
ITME - Day of the period.
J - Integer for iteration within period.
JDAY - Day on which maximum concentration occurred.
JG1 - Non-subscripted form of IG1.
JG2 - Non-subscripted form of IG2.
JMJ - Integer truncate of QADD
JR1 - Non-subscripted form of UR1.
JR2 - Non-subscripted form of UR2.
K - (K1 + K3) or iteration integer.
K1 - Deoxygenation rate coefficient (base e).
K2 - Reaeration rate coefficient (base e).
K3 - Rate constant for BOD removal by sedimentation or adsorption.
KMK - Integer truncate of QADD.
L - Length of reach in miles.
LA - Initial biochemical oxygen demand in mg./l.
LIMIT - Acceptable limit of a pollution parameter.
LK - Dummy iteration integer.
LO - BOD load at outlet of reach in lbs./day.
LOADT - Identifier of subroutine to compute BOD after T days.
LOC - Location of critical point within the reach in miles.
LRCH - BOD load after TF days.
LT - BOD load after T days.

LTF - BOD load after TF days.
MA - Multiplier for A.
MK1 - Multiplier for K1.
MK3 - Multiplier for K3.
MORGA - Multiplier for ORGA.
MP - Multiplier for P.
N - Iteration integer.
ND - Number of days in period.
NLIM - Non-conservative limit.
NM - NUM + 1.
NNN - Identifier of a reservoir reach.
NPT - Identifier of routine for reservoir identification.
NTTF - Day of period plus time of flow through reach.
NUM - Equation identifier for CADD function for reservoirs.
ORGA - BOD load added as a point source in lbs./day.
P - Rate of addition of BOD along the reach.
Q - Discharge out of a reach in cfs.
QAA - Coefficient of QADD function.
QAB - Constant of QADD function.
QADC - First breakpoint on CADD function.
QADC2 - Second breakpoint on CADD function.
QADD - Discharge directly into reach from surface runoff and groundwater.
QCRIT - Breakpoint on CADD function for reservoirs.
QOUT - Discharge out of a reach in cfs.
QRCH - Discharge in a reach in cfs.
S - Mean bed slope of the reach.
SAT - Saturation concentration of dissolved oxygen.
SKIP - Indicator of the existence or non-existence of a reach.
T - Time in days from start of sag.
TC - Time in days to critical sag.
TCR - Identifier of subroutine to compute TC.
TF - Time of flow through reach in days.
UR1 - Identity of an upstream reach.

UR2 - Identity of a second upstream reach (Tributary).

V - Average velocity of flow in ft./sec.

VA - Coefficient of V function.

VB - Exponent of V function.

ZA - Value of A.

ZK1 - Value of K1.

ZK3 - Value of K3.

ZORGA - Value of ORGA.

ZP - Value of P.

CRES LABORATORIES

Chemical Engineering Low Temperature Laboratory

Remote Sensing Laboratory

Electronics Research Laboratory

Chemical Engineering Heat Transfer Laboratory

Nuclear Engineering Laboratory

Environmental Health Engineering Laboratory

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